SEISMIC FULL-SCALE TESTS ON A 3D INFILLED RC FRAME

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ABSTRACT: Shaking table tests on a 3D infilled RC frame have been performed, as a part of a more complex research project, in the aim to study the effects of infill walls on RC frame seismic response. In particular a one-storey one-bay RC infilled frame, designed according to old seismic Italian Code and without any Capacity Design provision, is considered. As in Italian practice, infill walls were double panels without wall ties.

Test sequences included mono and bi-directional seismic actions while white noise tests were carried out to evaluate structural dynamic characteristics before and after each seismic test. Bare and infilled configurations were tested with a peak ground acceleration increasing level up to failure of infill panels with openings. A final so-called "semi-infilled" configuration was tested, i.e. after damaged panels with openings removal; two series of increasing seismic were experienced until a peak ground acceleration that was 60% higher than calculated collapse acceleration for prototype bare frame. Numerical evaluations of spectral parameters, based on pushover analyses with an original infill model, are shown and discussed.

KEYWORDS: masonry panels, full scale seismic tests, infilled RC frame, push-over analysis

1. INTRODUCTION

Some results of tests performed on a 3D infilled RC frame are discussed in the present paper. These tests were performed at the CEA of Saclay (France) by a research team of the Universities of Patras (Greece), Roma Tre and Chieti-Pescara (Italy) and are included in an Ecoleader European research program (Le Maoult 2005).
Ecoleader project includes four spatial full-scale frame structures. Three of them are two storey bare frames; they were designed and analyzed by other research groups in order to evaluate seismic response of a RC frames representative to old seismic design procedure. Same prototype structures are been used for an evaluation on new trend dissipative systems as Fluid Viscous Dampers, Antonucci et al. (2004). The last frame of the series is the current one, in this case a more traditional building system was detected: an infill frame with non load bearing hollow bricks. This chose in order to evaluate seismic response of a great part of existing buildings and possibility that a well done infill masonry panel could be an excellent tool for existing RC frames improvement.
For this reason the infill masonry design was with the aim to obtain a specimen representative of a real common typology of RC infilled frame buildings: double clay bricks panels of current quality. Infill panels are quite slender and have no links (like steel keys or other) with RC elements or wall ties. On the contrary mortar was designed in order to be an extremely high quality mortar that could be an innovative approach, considering that generally in this case both mortar and bricks are non engineering elements. Structural loads on specimen are arranged in order to reproduce the stress pattern in columns and foundations detected in two storey prototype.

2. SPECIMEN DESIGN

Structural design of specimen was carried out in order to obtain a prevalent vertical load carrying structure with low ductility and poor seismic reinforcement detailing. Infill panel typology was chosen in order to reproduce a commonly used one in Italian current practice. As said above normal clay bricks were used while high quality mortar was designed in order to control the influence of this component on orthotropic behavior of panel. The one-storey specimen is representative of the first storey of a shear–type prototype (previously tested at CEA Laboratory in a two-storey configuration). Prototype was characterized by poor strength, great lateral flexibility, low local and global ductility and uncontrolled strength hierarchy. The prototype frame was designed with a PGA = 0.07g, low strength (20 MPa) concrete and a high yielding strength (550 MPa) reinforcement. This is typical of 70's and 80's RC frame buildings (Biondi 2008).

A one-storey one-bay specimen geometry was: 4.00×4.00 m^2 plan, 3.80 m high, 260×260 mm^2 square columns, 400×260 mm^2 beams, 120 mm thick RC slab for first floor. Additional masses (30 tons) were fixed on the slab in order to simulate the second floor influence (global mass was 55 tons in infilled configuration). The RC structure is regular and symmetrical in the two directions while infill panels are symmetrical in one direction only. Two parallel single-leaf walls without openings (X direction) and two opposite walls with different openings (a 1.00×1.20 m^2 window, and a 1.00×2.10 m^2 door, Y direction) were made. Infill walls were double panels without wall ties. Bed and perpendicular joints were made with high strength site-made masonry mortar. Hollow non-structural bricks (horizontally perforated units with rendering keyways, 6 holes, 80×160×330 mm^3) for internal and semi-solid bricks (vertically perforated units with single grip hole, 120×120×250 mm^3) for external panels were used. Further aspects regarding specimen design, construction and instrumentation for tests are discussed in Albanesi et al. (2006-2008.a.b.c.), Candigliota et al. (2007).

3. SEISMIC TEST SEQUENCE

Tests were performed on 6 DOF shaking table at CEA Laboratory of Saclay. Three specimen configurations have been tested: bare frame (1st), undamaged infilled frame (2nd) and damaged infilled frame (3rd), Table 1. Horizontal time-histories were applied. Before and after each seismic test, white noise tests at low intensity (*PGA=0.05* g) were performed to evaluate specimen frequencies. It is to note that white noise intensity is quite the conventional elastic horizontal load used in the old-style seismic design of prototype structure $F_h = 0.07\ W_t$.

Table 1. Tests sequences: bare frame, infilled frame and damaged infilled frame.

Bare frame 1st test sequence	*PGA*
White noise X & White noise Y	0.05 g
Test 1: mono-directional **Ox** time history test	**0.10 g**
White noise X	0.05 g
Test 2: mono-directional **Oy** time history test	**0.10 g**
White noise X & White noise Y	0.05 g
Test 3: mono-directional **Ox** time history test	**0.15 g**
White noise X & White noise Y	0.05 g
Test 4: mono-directional **Oy** time history test	**0.15 g**
White noise X & White noise Y	0.05 g
Test 5: bi-directional **Ox&Oy** time history test	**0.10 g**
White noise X & White noise Y	0.05 g
Test 6: bi-directional **Ox&Oy** time history test	**0.15 g**

Damaged infilled PGA frame 3rd test sequence	*PGA*
White noise X	0.05 g
Test 1: mono-directional **Ox** time history test	**0.10 g**
White noise X	0.05 g
Test 2: mono-directional **Oy** time history test	**0.55 g**
White noise X	0.05g

Infilled frame 2nd test sequence	*PGA*
White noise X & White noise Y	0.05 g
Test 1: mono-directional **Ox** time history test	**0.10 g**
White noise X & White noise Y	0.05 g
Test 2: mono-directional **Oy** time history test	**0.10 g**
White noise X & White noise Y	0.05 g
Test 3: mono-directional **Ox** time history test	**0.20 g**
White noise X & White noise Y	0.05 g
Test 4: mono-directional **Oy** time history test	**0.20 g**
White noise X & White noise Y	0.05 g
Test 5: bi-directional **Ox&Oy** time history test	**0.10 g**
White noise X & White noise Y	0.05 g
Test 6: bi-directional **Ox&Oy** time history test	**0.20 g**
White noise X & White noise Y	0.05 g
Test 7: bi-directional **Ox&Oy** time history test	**0.30 g**
White noise X & White noise Y	0.05 g
Test 8: bi-directional **Ox&Oy** time history test	**0.45 g**

The 1st configuration was the bare frame (in Table 1, **Ox** and **Oy** tests are mono-directional x and y tests while **Ox&Oy** tests are bi-directional time history tests) and six seismic tests at increasing intensity (*PGA=0.10÷0.15 g*) have been performed in September 2004. The 2nd configuration was the RC infilled frame, four mono-directional and four bi-directional seismic tests with increasing intensity (*PGA=0.10÷0.45 g*) have been performed in December 2004. The 3rd configuration was the damaged infilled RC frame with infill panel in X direction. Two mono-directional seismic tests (*PGA=0.10, 0.55 g*) have been performed in January 2005.

In this paper attention will be focused on top acceleration and top displacements. Figure 1 shows RC frame instrumentation used for bare frame; in case of the infilled frame, diagonal displacements were recorded at each structure side. Acceleration has been recorded by means of accelerometers, top displacements, δ_{hxj}, are derived by means of diagonal displacement sensors at the top of column j as:

$$\delta_{hxj} = \frac{\delta_{ij}}{\cos\theta} \qquad \theta = \arctan\frac{h_j}{l_{ij}} \tag{3.1}$$

where δ_{ij} = diagonal displacement, l_{ij} = column axes distance in i-j plane, h_j = j^{th} column height at beam axis.
In order to control local stress distribution and structural interaction, RC columns-to-panel relative displacements were recorded by means a complete recording system. The instrumental apparatus on infill panels permitted to record masonry strains by means of two series of three deformometers (total length 430 mm, base length on masonry 300 mm; at the top, the medium and the bottom of the column) and three parallel strain gauges (on first three horizontal brick courses). In this paper two relevant aspects have to be pointed out: the first that a considerable, sometime unexpected, gap was detected between RC frame and infill panel, the second that, due to this gap, only for high seismic levels recorded relative displacements between RC structure and infill panels were "in-phase". This has to be outlined when numerical analysis is carried out and, probably, it is the reason of some underestimation of horizontal displacements that will be shown in the following chapters.

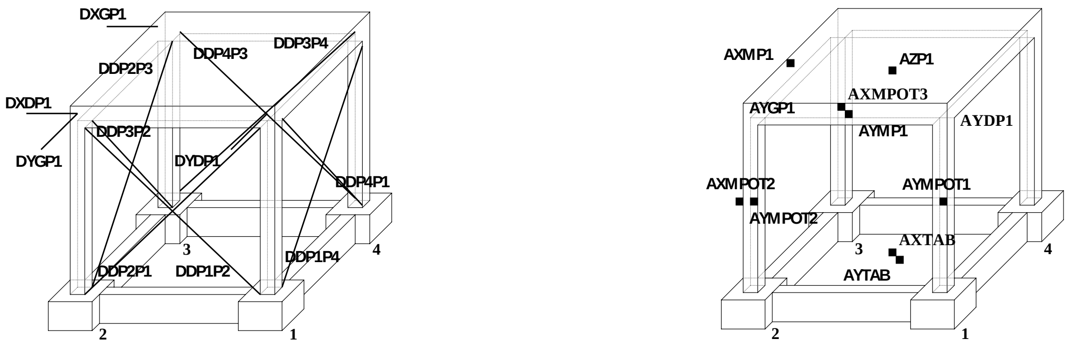


Figure 1. Displacement (a) and acceleration (b) sensors configuration on bare specimen.

4 DYNAMICAL TEST RESULTS

Test sequence on bare frame verified structural design and frame characteristics. The double structural symmetry was confirmed: similar elastic frequencies both in X and in Y directions (Table 2). Again seismic tests at *PGA=0.15 g* (X, Y or XY tests) didn't show frequency decay, i.e. relevant structural damage. It possible to suppose that RC elements are cracked while steel yielding isn't attained, according to linear elastic hypothesis and allowable stress design criteria. Infill panels increase global horizontal stiffness as stated by structural frequencies increase: almost 3 times in Y direction (panels with openings) and 2 times in X direction (panels without openings). This result in Y direction might be considered as very surprising due to the presence of panels with openings. Substantially, in the elastic range, panels with openings arranged as in this specimen (regular panel, accurate workmanship and symmetrical opening distribution with concrete lintel and with high quality masonry mortar) seemed to have a structural behavior very similar, if not better, of full infill panels.

Table 2. Measured first frequencies [Hz] after each seismic test for the 3 test sequences (Candigliota et al. 2007).

Bare frame (1st)			4 walls infilled frame (2nd)			2 walls infilled frame (3rd)		
test	f_x	f_y	*test*	f_x	f_y	*test*	f_x	f_y
	2.730	2.800		4.850	6.800		2.800	4.600
0.10g X	2.340	-	0.10g X	4.300	6.050	0.10g X	2.600	4.300
0.10g Y	2.340	2.240	0.10g Y	4.300	5.000	0.55g X	2.140	-
0.15g X	2.050	2.240	0.20g X	3.700	4.780			
0.15g Y	2.050	2.050	0.20g Y	3.700	2.600			
0.10g XY	2.050	2.050	0.10g XY	3.700	2.500			
0.15g XY	1.840	1.850	0.20g XY	3.220	2.440			
			0.30g XY	3.020	2.340			

As post-elastic range was reached, (2nd sequence, test 4: mono-directional Oy time history test, *PGA=0.20 g*) a sudden drop in Y stiffness (and consequently in Y frequency, f_y) was detected and global infilled frame stiffness, in Y direction, becomes almost equal to undamaged bare frame one. During the last test of this 2nd sequence (test 8) the internal wall with the door felt down. Notice that this internal panel was the thinner one (80 mm clay hollow bricks with horizontal holes) while falling was an in-plane failure. Structural behavior in full panel direction was much more regular even if, during low intensity tests, external walls without openings had an unexpected out of plane displacement (about 7 mm after 2nd configuration, test 2, *Oy 0.10 g*). Probably this out of plane was due to constructive phase but it didn't have any effect on structural response. In fact at the end of 2nd test sequence, the two full walls weren't significantly damaged and out of plane displacements were recovered; the RC frame wasn't much more damaged than after 1st test sequence.

Table 3. Bare frame: nominal, maximum base (a_{bx}-a_{by}) and top (a_{tx}-a_{ty}) accelerations.

Test	PGA/g	a_{bx}/g	a_{by}/g	a_{tx}/g	a_{ty}/g
Test 1: Ox	0.10	0.172	0.010	0.224	0.019
Test 2: Oy	0.10	0.012	0.137	0.020	0.297
Test 3: Ox	0.15	0.150	0.010	0.356	0.015
Test 4: Oy	0.15	0.005	0.169	0.012	0.484
Test 5: Ox&Oy	0.10	0.131	0.144	0.301	0.331
Test 6: Ox&Oy	0.15	0.206	0.205	0.379	0.430

Table 4. Infilled frame: nominal, maximum base (a_{bx}-a_{by}) and top (a_{tx}-a_{ty}) accelerations.

Test	PGA/g	a_{bx}/g	a_{by}/g	a_{tx}/g	a_{ty}/g
Test 1: Ox	0.10	0.209	0.034	0.386	0.056
Test 2: Oy	0.10	0.022	0.141	0.051	0.190
Test 3: Ox	0.20	0.260	0.027	0.499	0.068
Test 4: Oy	0.20	0.039	0.260	0.083	0.456
Test 5: Ox&Oy	0.10	0.132	0.107	0.227	0.178
Test 6: Ox&Oy	0.20	0.253	0.211	0.464	0.407
Test 7: Ox&Oy	0.30	0.329	0.318	0.682	0.532
Test 8: Ox&Oy	0.45	0.572	0.434	1.259	0.954

Table 5. Damaged infilled frame: nominal, maximum base (a_{bx}-a_{by}) and top (a_{tx}-a_{ty}) accelerations.

Test	PGA/g	a_{bx}/g	a_{by}/g	a_{tx}/g	a_{ty}/g
Test 1: Ox	0.10	0.128	0.025	0.257	0.038
Test 2: Ox	0.55	0.836	0.079	1.363	0.349

In Table 3 to Table 5, maximum accelerations are summarized: in particular nominal PGA, maximum base accelerations (a_{bx}-a_{by} on table in X and Y direction respectively), and top accelerations (a_{tx}-a_{ty} on beams) are shown. According to stiffness distribution in X and Y directions, a higher top acceleration in X direction is generally detected (obviously for infilled frame). As an example, top horizontal frame displacements measured on the infilled frame (4 walls) during test 8 (Ox&Oy, PGA=0.45 g) were: δ_{hx}=6.24 mm, (frame 1-2), δ_{hy}=10.82 mm (frame 2-3), δ_{hy}/δ_{hx}=1.73 if average (positive-negative) values are considered. At the same seismic stage, horizontal displacements recorded on external and internal walls in X and Y direction shown a great difference between full walls (C_e, C_i) and walls with opening (B_e, B_i). In the first case internal and external behavior is similar while in the second one the thinner internal panel had displacements much greater than the external one. For this reason if a strut model has to be used (Biondi et al. 2006) for a wall with openings, different (single or multiple) struts have to be defined for internal and external panels or an homogenization criterion has to be defined. At same stage it is possible to note a harmonic behavior between RC frame and infill panel: for *PGA=0.45 g* a regular compressive deformation is detected in panel masonry while tensile deformation is negligible, i.e. original gap between RC frame and masonry panel is uninfluential at these high seismic levels.

5. NON LINEAR PUSH-OVER ANALYSIS IN COMPARISON TO TEST RESULTS

Displacement time histories have been shown and discussed in Albanesi et al. (2008). The aim of this section is to evaluate structural frame response by means of non linear static pushover analyses, i.e. in terms of equivalent single-degree-of-freedom system. 2D pushover analyses have been performed by means non linear fiber models of both bare and infilled RC frame (Candigliota et al. 2007) including three different kinds of infill models: FEM, three struts and single equivalent strut (Albanesi et al. 2006.a.-b.). It is to note that a full infill has been considered in the models, so an overestimation of strength and stiffness could be expected in Y direction.

A comparison with experimental values is shown in Figure 2 in terms of force-displacement behavior. In this figure, the experimental data are single data defined as average values: so horizontal top displacement is the average of horizontal displacements, Eqn. (3.1), while experimental base shear is the product of mass by average top horizontal acceleration, a_{tx} and a_{ty}. The one bay-one storey infilled frame is quite really a SDOF, so the effective mass m^* is simply determined as global mass minus inferior half storey mass.

In the aim to control the step-by-step SDOF response, the classical push-over parameters [Albanesi et al. (2002), Dolšek et al. (2008)] are modified to consider at each seismic step yield force, F_y, and displacement, δ_y, of Eqn (5.1) as the current equivalent top force, F_F, and horizontal displacement, δ_h Eqn. (3.1),:

$$\delta_y = \delta_h \qquad\qquad F_y = F_F \tag{5.1}$$

Considering different push-over analyses: F_{FEM} nonlinear FEM analysis, F_{32a} - F_{32b} three struts non linear analysis (type a and b in Biondi et al. (2006.)), F_1 single strut model, the control parameters can be derived as:

$$\rho_T = \frac{T_{\exp}}{T_F} \qquad\qquad \rho_a = \frac{a_{t\exp}}{a_F} \tag{5.2}$$

$$F_F = K_F \delta_{h\exp} \qquad T_F = 2\pi\sqrt{\frac{m^*\delta_{h\exp}}{F_F}} \qquad a_F = \frac{F_F}{m^*} \qquad S_{eF} = S_{eEC8}\left(T_F\right) \tag{5.3}$$

In Table 6÷Table 8 experimental to theoretical ρ_T and ρ_a ratios are shown according to Eqn. (5.2)-(5.3) where subscripts h, exp, t, F seem respectively horizontal, experimental, top and determined by means of push-over analysis. A good prediction of bare frame force-displacement is shown in Figure 2 (left).

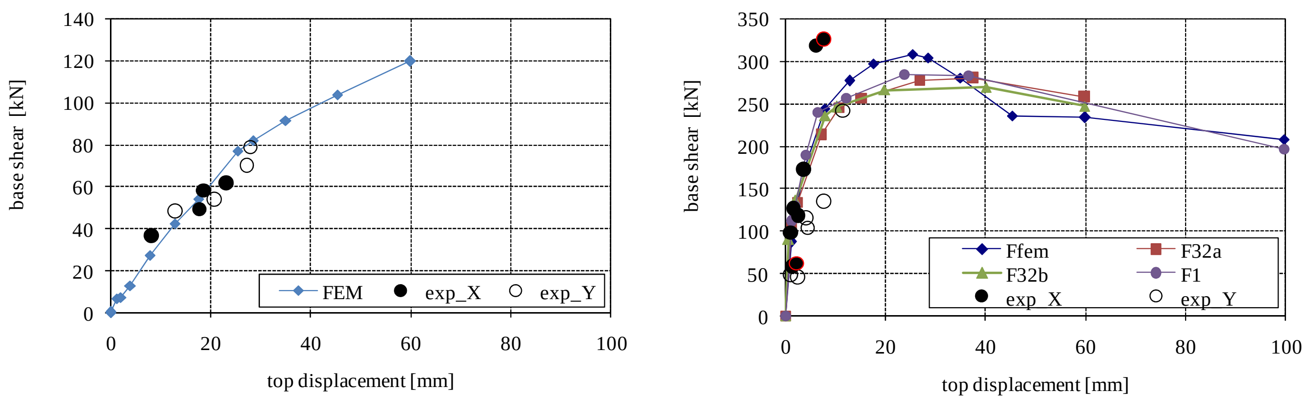


Figure 2. Capacity curves of bare (left) and infilled (right) frame for different kinds of infill models

In the case of infilled frame, the 2D pushover analysis performs well again in the elastic range with an evident stiffness overestimation, above all in Y direction where a full panel model is used for panels with openings.

Peak strength in X direction is well defined even if with an evident secant stiffness underestimation. In particular if experimental force-displacement values are respectively {318 kN, 6.17 mm} and {325 kN, 7.67 mm} for Test 8 [*Ox&Oy 0.45 g* 2nd series] and Test 2 [*Ox 0.55 g* 3rd series (red bordered dots in Figure 2 for this 3rd series)], peak values are {308 kN, 25.41 mm} and {279 kN, 26.80 mm} for F_{FEM} and F_{32a} analysis with respectively 5.50% and 16.40% of peak strength underestimation in respect to Test 2 3rd series.

Basing on these capacity curves and considering secant stiffness, it is possible to calculate the equivalent SDOF period corresponding to experimental horizontal displacements, Eqn. (5.3). Results are shown in Table 6÷Table 8, Figure 3÷Figure 4. As obvious period ratios ρ_T are practically equal to the unity for bare frame, Figure 3 (left), while for the infilled frame some discrepancies can be outlined: damage sequence is very similar in the test period plane, experimental and theoretical slope coincides but test periods are almost twice than theoretical ones.

The key question at this point is to evaluate if this stiffness overestimation is due to an overestimation of infill panel mechanical characteristics or it is due to an incorrect evaluation of coupling displacement between RC frame and infill panel. Considering that the assumed hypothesis for orthotropic masonry behavior (Biondi et al. (2006)) has been detected as conservative, i.e. underestimates in-plane stiffness of masonry panels (Albanesi et al (2008.c)), the second option appears as correct.

Table 6. Bare frame: comparison between experimental to theoretical period, ρ_T, and acceleration, ρ_a, ratios

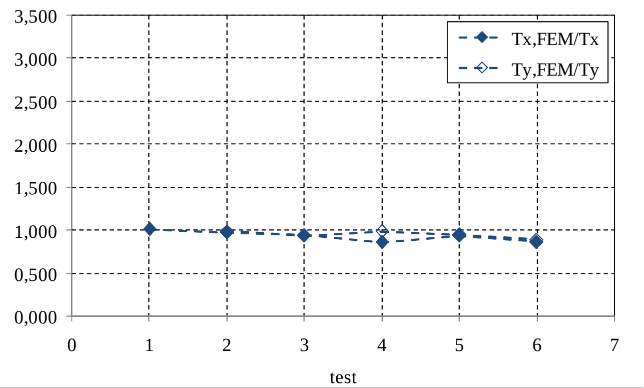
	period ratios		acceleration ratios			
	F_{FEM}		$EC8$		F_{FEM}	
Test	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir
Ox.0.10g	1.009	-	2.208	-	0.790	0.811
Oy.0.10g	0.973	0.991	1.750	1.325	1.396	0.892
Ox.0.15g	0.936	0.930	1.214	1.925	1.012	1.675
Oy.0.15g	0.855	0.985	1.120	1.001	3.330	1.007
Oxy.0.10g	0.931	0.947	1.249	1.252	1.148	1.183
Oxy.0.15g	0.861	0.886	1.565	1.370	1.121	1.111

Table 7. Infilled frame: comparison between experimental to theoretical period, ρ_T, ratios

	F_{FEM}		F_{32a}		F_{32b}		F_1	
Test	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir
Ox.0.10g	0.507	0.713	0.467	0.658	0.440	0.508	0.452	0.636
Oy.0.10g	0.507	0.589	0.467	0.544	0.361	0.504	0.452	0.526
Ox.0.20g	0.466	0.563	0.445	0.520	0.426	0.401	0.428	0.503
Oy.0.20g	0.436	0.397	0.402	0.415	0.311	0.405	0.389	0.383
Oxy.0.10g	0.455	0.342	0.429	0.337	0.412	0.327	0.520	0.325
Oxy.0.20g	0.447	0.383	0.443	0.401	0.430	0.390	0.425	0.368
Oxy.0.30g	0.449	0.423	0.470	0.445	0.457	0.432	0.436	0.421
Oxy.0.45g	-	-	-	-	-	-	-	-

Table 8. Infilled frame: comparison between experimental to theoretical acceleration, ρ_a, ratios

	$EC8$		F_{FEM}		F_{32a}		F_{32b}		F_1	
Test	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir
Ox.0.10g	1.553	1.559	0.815	0.949	0.957	1.114	1.082	1.867	1.023	1.191
Oy.0.10g	1.237	2.126	1.601	1.584	1.879	1.859	3.150	2.162	2.009	1.987
Ox.0.20g	1.500	1.123	0.824	1.115	0.904	1.310	0.985	2.195	0.976	1.400
Oy.0.20g	1.358	1.636	2.450	1.540	2.877	1.407	4.822	1.484	3.075	1.659
Oxy.0.10g	1.676	1.723	1.683	2.912	1.896	3.000	2.051	3.193	1.289	3.236
Oxy.0.20g	1.564	1.490	1.152	1.790	1.171	1.639	1.245	1.725	1.275	1.939
Oxy.0.30g	1.386	1.718	0.995	1.793	0.908	1.623	0.959	1.723	1.058	1.812
Oxy.0.45g	0.522	0.524	0.672	1.112	0.618	1.025	0.646	1.032	0.739	1.058



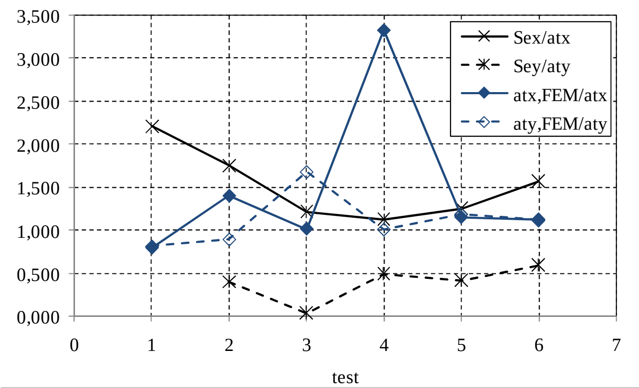


Figure 3. Bare frame: experimental-to-estimated period, ρ_T (left), and top acceleration, ρ_a (right), ratios

In order to evaluate effectiveness of push-over analysis, spectral components of response have been analyzed.

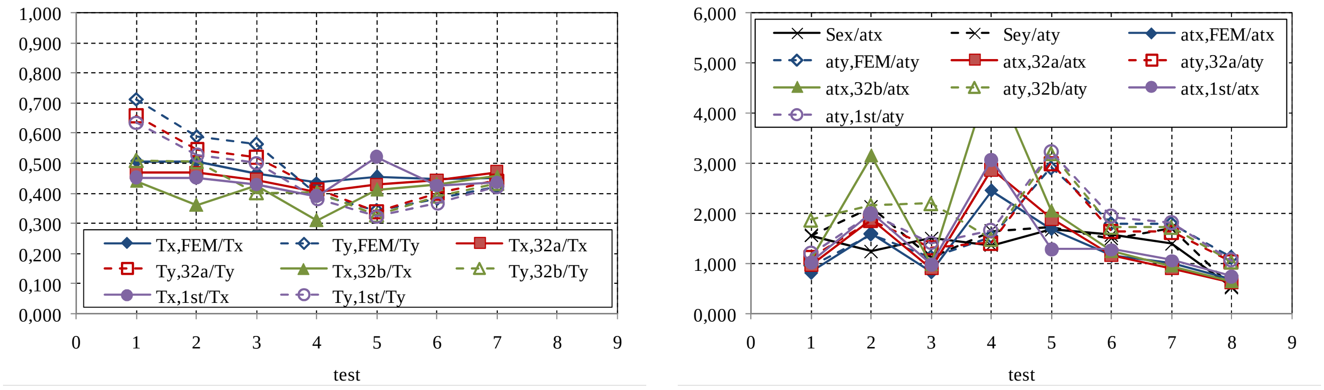


Figure 4. Infilled frame: experimental-to-estimated period, ρ_T (right), and top acceleration, ρ_a (left), ratios.

Horizontal time-histories for test sequences were designed as EC8 spectrum compatible for a C soil, Candigliota et al. (2007). According to EC8 spectrum shape, the elastic response spectrum at T=0 is $S \cdot a_g$ and the amplification factor is equal to $2.5 \cdot \eta$ where S=soil factor (=1.15 for type C soil), η=damping correction factor (=1 for 5% viscous damping) and a_g is the peak ground acceleration (PGA) factor on type A ground.
Assuming that structural response is in the constant spectral acceleration branch, i.e. $T \leq T_C$ as confirmed in Table 2, amplification factors can be defined as corresponding top acceleration-to- base acceleration ratios:

$$A_x = \frac{a_{tx}}{a_{bx}} \qquad A_y = \frac{a_{ty}}{a_{by}} \tag{5.4}$$

and are shown in Figure 3 and Figure 4 for both bare and infilled frame. In these figures all values are considered and a large scattering can be detected. If only values related to seismic direction are considered, i.e. only a_{tx} or a_{ty} respectively for Ox or Oy tests, this scattering disappear for bare frame, Table 3.
Behavior factors can be defined both as 2.50 times the inverse of amplification factors, i.e. experimentally determined, or as the corresponding elastic response spectrum-to-top acceleration ratios, i.e. theoretically determined for spectral component, where S_{ex} and S_{ey} are defined in (5.3):

$$q_{ax} = 2.50\frac{a_{bx}}{a_{tx}} \qquad q_{ay} = 2.50\frac{a_{by}}{a_{ty}} \tag{5.5}$$

$$q_{Sx} = \frac{S_{ex}}{a_{tx}} \qquad q_{Sy} = \frac{S_{ey}}{a_{ty}} \tag{5.6}$$

This comparison is shown in Figure 5 in terms of frame period; it is possible to note that amplification factors (square marks) in X (full marks) and Y (empty marks) directions have quite different values but the same trend.

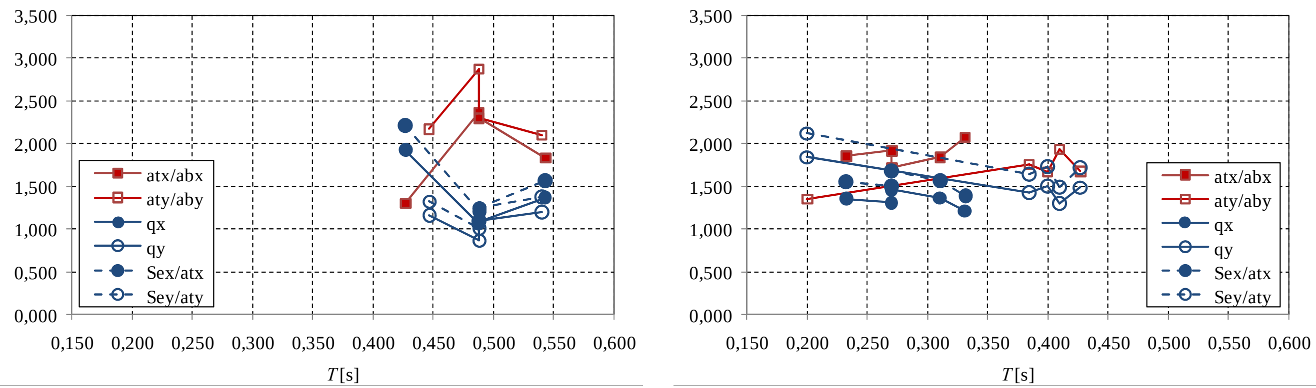


Figure 5. Amplification and behavior factors of the bare (left) and infilled (right) frame.

In the same Figure 5 behavior factors (circle marks) in X (full marks) and Y (empty marks) directions are defined both as in Eqn. (5.5) (solid line) and as in Eqn. (5.6) (dashed lines).

It is possible to note that theoretical to experimental values show the same trend and quite similar values if average values are considered: $\overline{q}_{Sx}/\overline{q}_{ax}=1.15$ and $\overline{q}_{Sy}/\overline{q}_{ay}=1.14$ for bare frame, $\overline{q}_{Sx}/\overline{q}_{ax}=1.07$ and $\overline{q}_{Sy}/\overline{q}_{ay}=1.06$ for infilled frame, $\overline{q}_{Sx}/\overline{q}_{ax}=1.15$ and $\overline{q}_{Sy}/\overline{q}_{ay}=1.15$ for damaged frame.
According to that it is possible to conclude that push-over analysis can permits a correct interpretation of SDOF shaking table response if a correct model of RC frame to infilled frame interaction is defined. In particular the presence of a gap between RC frame and infill panel has a relevant role in stress transmission between the two components. Probably this gap has to be modeled in numerical analysis.

6. CONCLUSIONS

Seismic tests on a full scale RC infilled frame specimen have been performed and highlight a significant influence of infill walls, even with openings and high slenderness, on RC frame structural dynamic response.
This influence appears to be relevant not only for low seismic actions but also in case of strong earthquake; in particular the tested specimen, designed for a low seismic action (*PGA*=0.07 *g*), was able to survive to a very strong earthquake (*PGA*=0.55 *g*) without any relevant structural damages. This can be partly related to infill walls which were built with a high quality mortar. Full panels and panels with door and window openings showed similar structural behaviors at low seismic actions; only for high seismic levels panels with openings performed in a fragile way mainly due to the thinner panels.
2D non linear static pushover analyses were performed and dynamic response in terms of periods and maximum expected accelerations was evaluated. The pushover analysis gives a very good prediction of the bare frame dynamic response which was practically linear in both X and Y directions as expected due to its symmetrical configuration. Good results were also obtained for the infilled frame in the elastic range even if peak strength in X direction was underestimated while, due to test nature, wasn't possible to evaluate softening branches.
Finally even if an overestimation in numerical stiffness evaluation was detected for low seismic action, if spectral parameters are considered a good accuracy can be obtained. In particular behavior factors comparison between theoretical provision and experimental data, shown a discrepancy lesser than 15%.

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