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NUMERICAL NON-LINEAR SIMULATION OF THE IN-PLANE BEHAVIOUR OF R/C FRAMES WITH MASONRY INFILLS UNDER SEISMIC-TYPE LOADING

George C. Manos [1], Vassilios J. Soulis [2]

[1] *Emeritus Professor, Department of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, Greece*
gcmanos@civil.auth.gr

[2] *Lecturer Department of Civil Engineering, Technical Educational Institute of Piraeus, Piraeus,*
vassilios_soulis@yahoo.com

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Abstract. *A non-linear 2-D numerical simulation is presented that can capture the in-plane hysteretic behaviour of reinforced concrete (R/C) frames with masonry infills when they are subjected to combined vertical and cyclic seismic type horizontal loads. The effectiveness of this simulation was validated extensively in the past by comparing numerical predictions with results from a series of pseudo-dynamic tests whereby a number of 1:3 scale, one-bay, one-storey R/C frame specimens, including relatively weak masonry infills, were subjected to combined vertical and cyclic horizontal seismic-type loads under controlled laboratory conditions. The numerical modelling of the surrounding R/C frame included its flexural non-linear behaviour by simulating the development of plastic hinges either at the two ends of the R/C beam and/or at the top and toe of the R/C columns. All geometric and mechanical characteristics of the concrete and the longitudinal reinforcement of the R/C cross-sections were considered in detail. Sufficient shear reinforcement was assumed for both R/C beam and columns capable of prohibiting shear mode of failure. However, observations of failure modes for R/C infilled frames with columns under-designed in shear have shown that, when subjected to earthquake loads, the development of shear failure for the columns is an additional realistic scenario. The first part of the work presented here is investigating such an interaction of masonry infill with the R/C frame simulating both flexural and shear modes of failure for the R/C columns. This numerical simulation retains the non-linear behaviour of the masonry infill itself as well as the non-linear behaviour at the contact area where the masonry-infill and the surrounding frame. The second part of the present work examines the influence of the foundation deformability for such multi-storey infilled frames having an increased in-plane stiffness due to the presence of the masonry infills.*

1 INTRODUCTION

An extensive experimental and numerical study ([3], [4], [5] ,[6], [7]) was carried out at the laboratory of Strength of Materials and Structures, Aristotle University, in order to examine the in-plane seismic behaviour of infilled R/C frames. This study as well as previous studies ([1], [2]) demonstrated that the stiffness and strength of the masonry infilled, single-storey, R/C frames can be affected by various non-linear mechanisms such as:

a) The non-linear behaviour of the R/C frame including flexural limit state (plastic hinges) and shear limit state.

b) The non-linear mechanisms that develop at the interface between the R/C frame and the masonry infill.

c) The level of non-linear deformations that develop within the masonry infill itself.

In all these cases it was reported that the frames developed plastic hinges at their R/C beam and columns. All these have been simulated successfully, as is shown in section 2. In addition, it is also demonstrated that through a certain modification to this numerical approach proposed by Soulis [1] the shear limit state of the R/C columns at predefined locations can also be simulated. In order to validate this proposed modification the experimental study conducted by Sariyiannis is utilized [8]. He subjected a number of one-storey one-bay 1/3 scaled infilled frames to axial forces at the columns as well as horizontal seismic type cyclic forces at the level of the beam. Nine repaired infilled frames and two repaired bare frames were examined in this study. These specimens were produced by repairing damaged 1/3 scaled R/C infilled frames tested previously by Stylianides [3]. These original virgin frames had a non-reinforced brick-masonry infill panel which was simply connected to the surrounding R/C frame by a continuous mortar joint. After this initial test sequence was completed the damaged frames were repaired in order to be tested again [8]. As expected, the type of failure of the columns depended on the type of failure of the masonry infills. In a number of specimens tested by Sariyiannis [8] shear sliding of the infill along a horizontal mid-height mortar joint or corner crushing of the masonry infill was accompanied by the shear failure of the R/C columns. Two of these masonry infilled frame specimens, namely F1NR and F8NR [8], failed as a result of corner crushing of the infill and shear failure of the column. The masonry infilled frame F8NR is employed in the current paper in an effort to validate the proposed modified numerical simulation of masonry infilled R/C frame that can also realistically simulate this shear type of failure.

Stavridis [9], also carried out an experimental program of quasi-static tests with single-bay, single-storey, masonry-infilled, non-ductile RC frames, which were scaled sub-assemblages of a prototype structure. These tested specimens were 2/3-scaled models of such frames in a prototype building with design details representative of those used in the construction practice in California during the 1920’s [9]. Two of these specimens (referred to as CU1 and CU2) were single-storey, single bay masonry infilled frames that were tested with quasi-static cyclic loads [9]. A third specimen represented a three storey frame with two bays; this was tested on a shake table at UCSD [9]. After reaching the peak load, the specimen CU1 developed diagonal shear cracks in the concrete columns as an extension of the dominant cracks in the masonry infill. The same failure pattern was reported for the CU2 specimen with a window opening. Again, one of the columns in the CU2 specimen had a major shear crack which developed close to mid-height of the column. Koutromanos et al. [10], demonstrated the ability of a proposed nonlinear finite element model to capture the response of masonry-infilled reinforced concrete frames under cyclic loads. Diffused cracking and crushing in concrete and masonry are simulated by a smeared-crack continuum model, while dominant cracks as well as masonry mortar joints are modeled with a cohesive crack interface model.

The interface model adopts an elasto-plastic formulation to describe the mixed-mode fracture of concrete and masonry. This model accounts for cyclic crack opening and closing, reversible shear dilatation, and joint compaction due to damage. The constitutive models have been validated successfully with experimental data and applied to the dynamic analysis of a three-storey, two-bay, masonry-infilled, non-ductile, reinforced concrete frame, which was tested on a shake table [9]. The plane-stress smeared-crack model was used for the simulation of the continuum elements representing masonry units and concrete elements. The numerical simulation presented by Koutromanos et al. [12] can accurately reproduce the load–displacement response, crack patterns, and failure mechanisms of the infilled frames including the shear cracking of infill and the surrounding frame.

Valid numerical models of multi-storey infilled frames were investigated in the past by many researchers (see relevant review in [3] and [4]). The numerical simulation proposed by Soulis [1] was used to simulate the behaviour of three-storey structural formations including masonry infills; in particular a multi-storey planar R/C frame structure, which was constructed and tested at the University of California, Berkeley by Klingner and Bertero [11] was examined. Reasonably good agreement was observed between the numerical results and the experimental measurements regarding the hysteretic behaviour of the "bare", and infilled three-storey specimens. In the second part of this paper the numerical simulation of the behaviour of a 6-storey masonry infilled R/C frame structure located at the Volvi-Greece European Test Site for Earthquake Engineering [12] is examined employing an elastic dynamic analysis and a non-linear "push over" analysis. Six basic structural configurations are studied including a "Bare" structure (only the R/C frames without any masonry infills) and various formations that resulted with the addition of masonry infills for this specimen of a simple R/C multi-storey building. These structural formations that were studied numerically correspond exactly to this 6-storey 1/3 scaled model building during a period of 10 years, from 1994 onwards. A large number of low-amplitude dynamic tests have been conducted in-situ over an extended period, mainly from 1995 till 2006. By combining this large volume of dynamic response measurement data from the various in-situ low-vibration sequences, the most important mode shapes and eigen-frequencies were identified. This six-storey masonry infilled structure is numerically simulated in section 3. Initially, its dynamic behaviour is predicted utilizing the same simulation technique adopted in section 2 assuming, however, linear material properties and different foundation conditions. Next, a non-linear "push over" numerical analysis is performed in order to identify numerically its bearing capacity and the sequential appearance of various failure modes as predicted by this numerical simulation. In this "push over" analysis the soil-foundation conditions of the six-storey masonry infilled R/C frame structure is examined assuming either rigid or relatively flexible contact interface.

Frame Code name	Vertical load on Columns (KN)	Technical description of masonry infill	Masonry Infill thickness (mm)	Technical description of the interface between frame and infill	Longitudinal reinforcement ratio (ρ)
F8NR **Repaired** [8]	80	Infill with mortar **O** reinforced with reinforced plaster, without transverse reinforcement	83	mortar **O** thickness **10mm** The reinforced plaster is in contact with the surrounding frame	1.01%
CU1[9]	156	Infill with mortar **N**	95	mortar **N** thickness **10mm**	1.00%
CU2[9]	156	Infill with mortar **N**	95	mortar **N** thickness **10mm**	1.00%

Table 1 Outline of all specimens for the 1st and 2nd group of specimen

2 THE NUMERICAL SIMULATION OF THE BEHAVIOUR OF SINGLE-STOREY SINGLE-BAY MASONRY-INFILLED R/C FRAMES

The present work studies the interaction of masonry infill with the R/C frame including in this numerical simulation the development of flexural (plastic hinge) and/or shear limit state for the surrounding R/C frame. Three scaled R/C infilled frame specimens, one of them, namely specimen F8NR, tested by Saryiannis [8] and two of them, namely CU1, CU2, tested by Stavridis[9], are utilized for the verification of the proposed numerical simulations. In all these cases the numerical response predictions are compared with the corresponding experimental results. Brief information on the selected masonry infilled R/C specimens is listed in table 1 and figures 1 and 2. The influence exerted by the interface mortar joint between the masonry infill and the surrounding frame was also examined here [3], [4], as was also done in both studies by Thauampteh [2] and by Soulis[1]. Tables 2, 3 and 4 list the mechanical properties of the materials used in the construction of the specimens.

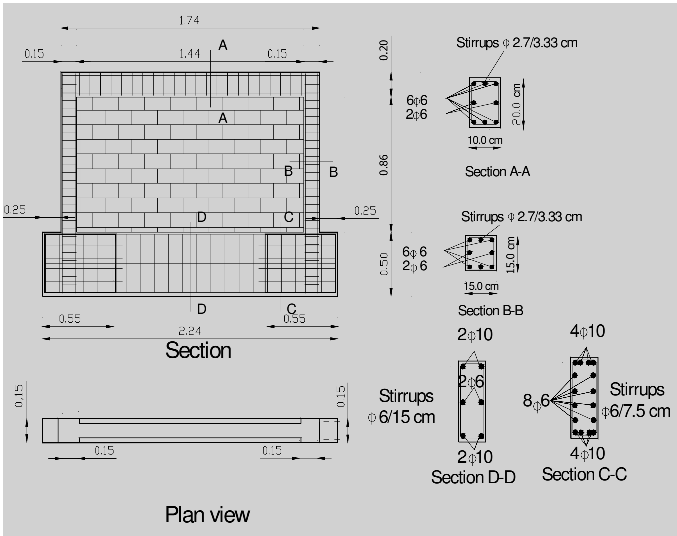


Figure 1. Masonry infilled R/C frame F8NR specimen and design details [8].

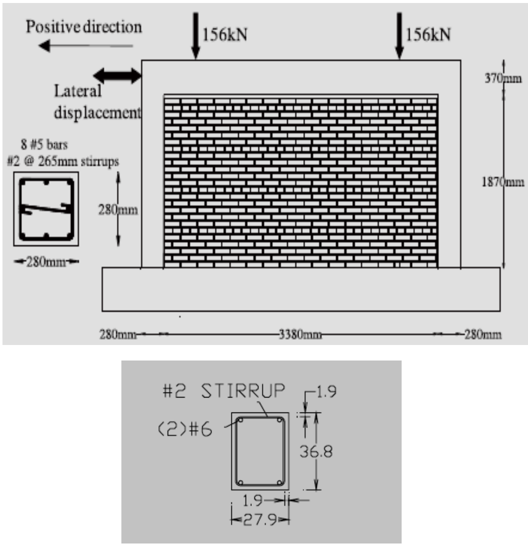


Figure 2. Masonry infilled R/C frame CU1 specimen and design details [9].

Masonry infill	Compressive strength of masonry (N/mm^2)	Shear strength of masonry diagonal compression (N/mm^2)	Compres. strength of masonry units (N/mm^2)	Compres. strength of concrete (N/mm^2)	Compres. strength of mortar cylinders (N/mm^2)	Compress. strength of cement plaster (N/mm^2)	Compres. strength of Injected Epoxy Resin (N/mm^2)
Reinforced infill F8NR [8]							
Infill with mortar **O**, reinforced with reinforced plaster without transverse reinforcement	2.12	0.64 (corner crushing)	5.50	25.9	2.7	11.3	94
Virgin infill CU1[9]							
Infill with mortar **N**	24.3		49	29,5	9.2		
Virgin infill CU2[9]							
Infill with mortar **N**	24.3		49	29,5	9.2		

Table 2**:** Strengths of masonry infills and concrete used in the specimens [8],[9]

A/α	Yield stress f_{sy} (N/mm^2)	Ultimate strength f_{su} (N/mm^2)	Strain at yield ε_{sy} (%)	Strain at ultimate stress ε_{su} (%)	Young Modulus (N/mm^2)
Reinforced infill F8NR [8]					
Φ5.5	348	457	0.174	18.0	2X10^5
Φ2.7stirrups	271	395	0.135	19.0	2X10^5
Virgin infill CU1[9]					
Φ#5 15.9mm	472	752	-	9.0	
Φ#2 6.4mm stirrups	431	472	-	13.0	

Table 3: Tensile strength of the reinforcement used in the specimens [8],[9]

A/a	Simulation of joint interface between frame and infill	**E** Young Modulus (N/mm^2)	**G** Shear Modulus (N/mm^2)	f_k Measured Compressive Strength of mortar (N/mm^2)	f_{tn} Assumed Tensile Strength of mortar (N/mm^2) (as % of f_c)	**τo** Local bond shear strength of mortar (N/mm^2)	**μ** friction coefficient
1	**O** mortar F8NR	540	234	2.70	0.27(10%)	0.31	0.58
2	**N** mortar CU1	1350	587	9,00	0.50		0.90
3	**N** mortar CU2	810	352		0,40		0.90

Table 4: Mechanical properties of the mortar joint located between the infill and the surrounding frame (mortar type H, N) [3], [4]

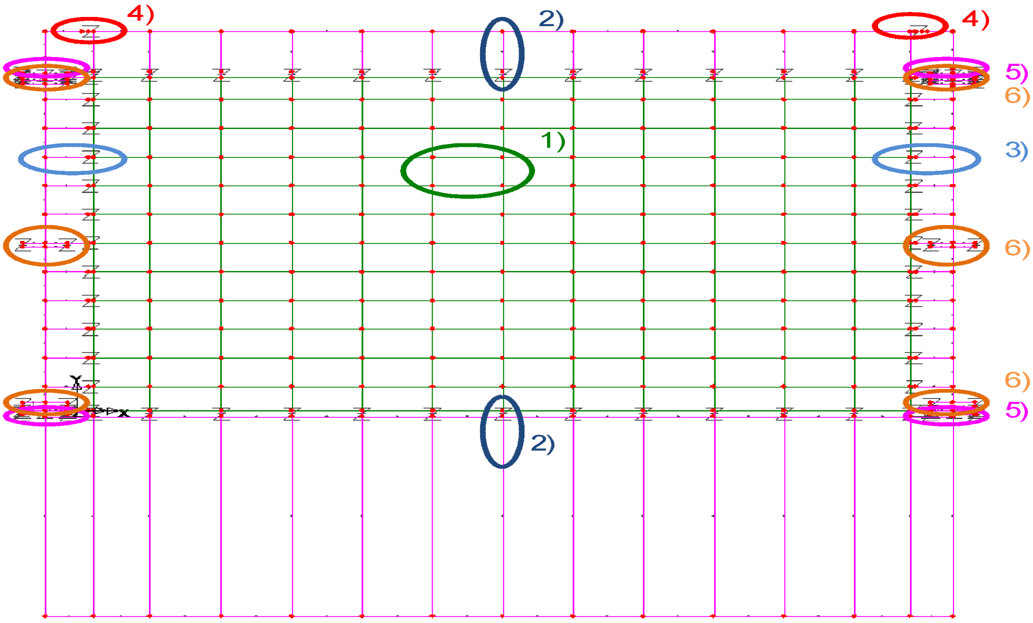


Figure 3. Features of the non-linear numerical simulation of a single-bay single-storey masonry infilled R/C frame

2.1 Simulation of the single storey masonry infilled R/C frame

An extensive study of various numerical simulations of the behaviour observed for masonry infilled R/C one-bay one-storey frame specimens tested by Thauampteh [2], by Stylianides [5], by Valiasis [6], and by Yasin [7] was included in the work by Soulis[1] together with an extensive validation process, utilizing the results of all these experimental studies ([2], [5], [6], [7]). Here a modified numerical model similar to the one presented before is examined that incorporates the simulation of the formation of shear limit state in predefined locations along the height of the two columns of such R/C specimens. The features of the numerical simulation are presented in figure 3 and in more detail in figures 4 and 5. These features are the same as the ones proposed by Manos, Soulis & Thauampteh before ([3], [4]) with the addition of the possibility of shear limit state at predetermined locations along the height of the R/C columns of the single storey R/C frame. In figure 3 these non-linear mechanisms are denoted as: (1) corresponding to the non-linear behaviour of the masonry infill itself, (2) the infill-beam interaction at the contact region, (3) the infill-column interaction at the contact region (4) the formation of a plastic hinge at the end of a beam, (5) the formation of a plastic hinge at the top or bottom of a column and finally (6) the formation of a shear limit state along the height of a column. These non-linear mechanisms are also depicted in some detail in figure 4 (non-linear mechanisms (1), (2), (3) and (4) and in figure 5 (non-linear mechanisms (5) and (6)).

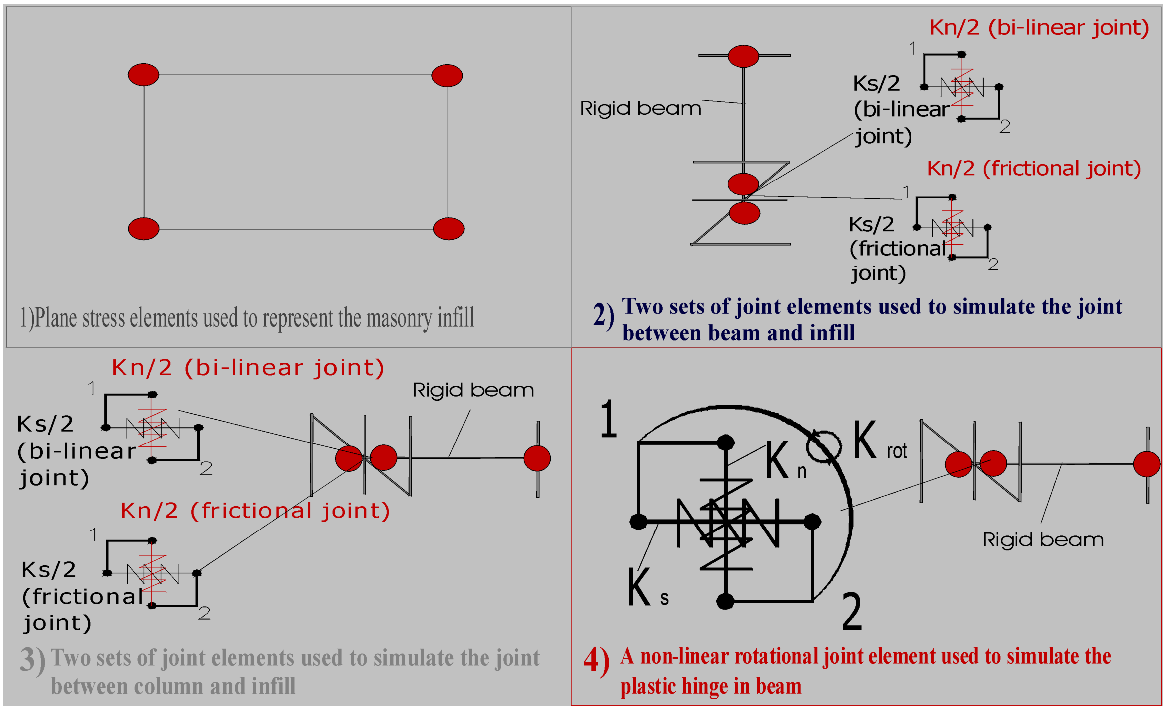


Figure 4. Details of the non-linear numerical simulation for the infill, the interface between infill and R/C frame and for the plastic hinge in the R/C beam.

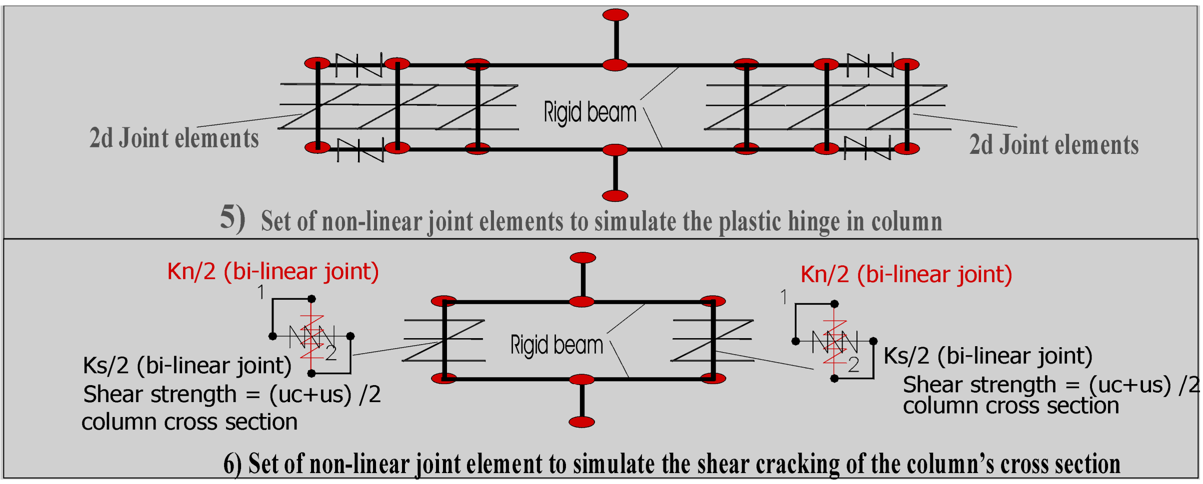


Figure 5. Details of the non-linear numerical simulation for the plastic hinge in the R/C column and for the shear limit state also at the column.

All these non-linear mechanisms are numerically simulated with a combination of rigid zones and non-linear joint elements (figures 4 and 5) with properties based on the geometric data, the material properties of the constituent materials (masonry, mortar, concrete, longitudinal steel) and the relevant structural detailing. Each one of the numerical simulations of the specific non-linear mechanisms (1) to (5) has been validated separately by Soulis [1] and Manos ([3], [4]) utilising relevant laboratory measurements obtained by Thauampteh [2]. Subsequently, this validation was also complemented by including all these non-linear mechanisms in the numerical simulation of a number of one-bay one-storey laboratory specimens by comparing the predicted cyclic behaviour for these specimens with observed performance ([1], [2], [3], [4], [5], [6], [7]). The additional shear limit state non-linear mechanism, represented by detail (6) in figure 5, is numerically approximated again by a combination of rigid zones and non-linear joint elements with properties based again on the geometric data of the column cross section, the material properties of the constituent materials (concrete, transverse steel), and the relevant structural detailing. The shear limit state was obtained by calculating the shear capacity for a column cross-section as specified by equations 1, 2 and 3. The location of these non-linear shear limit state zones is based on observed behaviour and accumulated experience for each given case. For the studied specimens three predefined locations along the height of each column were selected capable of developing this shear limit state. Two of these locations are at the ends (top and bottom) of each column in serial sequence with the non-linear mechanism simulating the flexural plastic hinges in these locations. An additional shear limit state mechanism was also placed at mid-height of each R/C column. Two elasto-plastic non-linear 2-D joint elements are used to simulate the shear post-elastic behaviour of a given R/C column cross section by considering also the corresponding column transverse reinforcement (figure 2 and detail No 6 in figures 3, 4 and 5). As per ACI 318:2008 [12] the total shear V_u resisted *(107KN)* by the column's cross section is carried by two parts; the shear resisted by the concrete and the shear resisted by the reinforcement. As already mentioned, the properties of these non-linear 2-D joint elements are specified utilizing the given structural details and equations 1 to 3. Independent numerical tests were carried out employing simple structural problems in order to verify that this way of simulating numerically the shear limit state mechanism responds in the expected way. Next, this numerical approach was applied to the infilled frame specimens included in table 2.

$$V_u = V_c + V_s \tag{1}$$

• Shear resisted by the concrete part of the cross section V_c

• Shear resisted by the transverse steel reinforcement V_s

For normal weight concrete,

$$V_c = \delta \times \left(0{,}17 \times \sqrt{f'_c}\right) \times b \times d \;\; (where, \delta = 1 + \frac{P_u}{14A_g}) \tag{2}$$

$$V_S = \frac{A_{SV} \times f_{yh} \times d}{s_v} \qquad for\ transverese\ reinforcement \tag{3}$$

f_c =cylindrical compressive strength of concrete
b = width of member
P_u = axial force normal to cross section
A_g =gross cross sectional area of concrete

A_{sv} = area of transverse steel reinforcement expected to be effective in resisting the expected shear mode of failure
d = Effective depth of member
s_v = spacing of stirrups
f_{yh}= yield stress of stirrups

2.2 Validation of the proposed numerical simulation

Figures 6a, 6b and 6c depict a comparison between the predicted behaviour, in terms of envelope curves resulting from a monotonic type of loading on the adopted numerical simulation and the corresponding envelope curves resulting from the relevant experiments with the masonry infilled frame specimens F8NR and CU1, CU2.

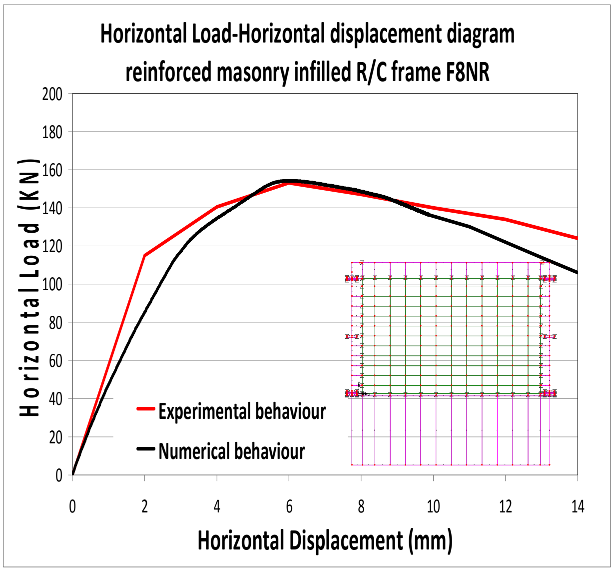


Figure 6a: Comparison of envelope curves for masonry infilled model frame F8NR

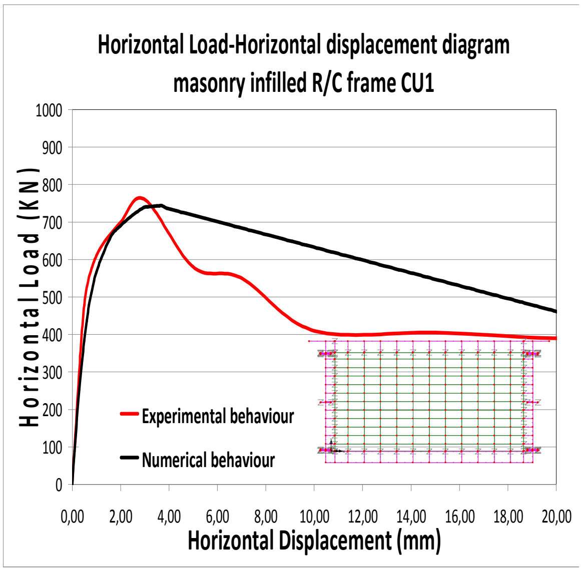


Figure 6b: Comparison of envelope curves for masonry infilled model frame CU1

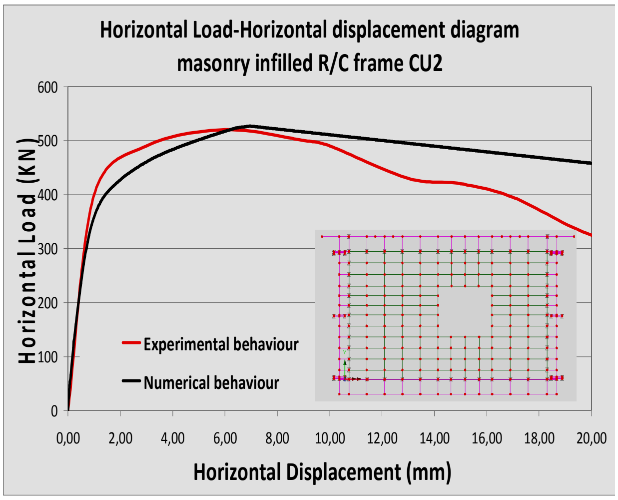
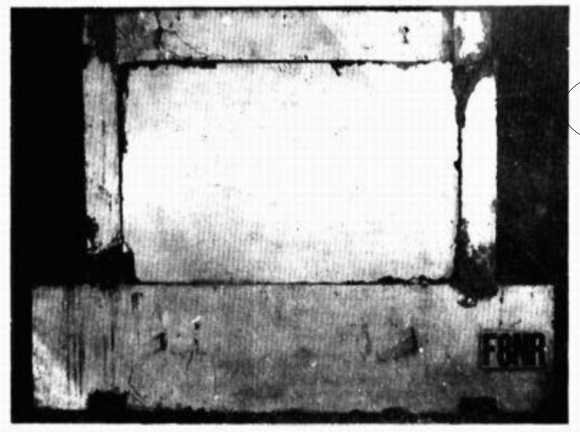
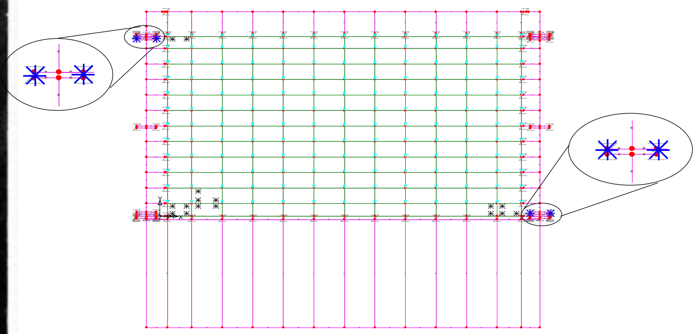


Figure 6c: Comparison of envelope curves for masonry infilled model frame CU2

The deformations to masonry infill observed during the tests and the numerically-predicted non-linear limit state is presented for frames F8NR (figures 7a and 7b), CU1 (figures 7c and 7d), CU2 (figures 7e and 7c). The observed damage patterns of the masonry infill were well approximated by the proposed numerical simulation. The damage pattern of F8NR masonry infilled R/C frame reported experimentally, consisted of corner crushing of the reinforced masonry infill corners in the R/C column and beam joint and a shear cracking in top right column(figure 7a).

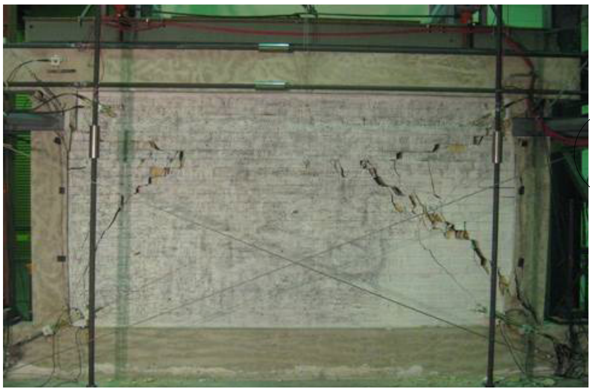
Reasonably good agreement can be seen in the damage pattern predicted numerically except that shear limit state is observed both in the top left and the bottom right column (figure 7b). The damage pattern that was observed for masonry infilled frame CU1 includes diagonal/sliding cracks in the infill panels and shear cracks on the top left column and shear cracks in the bottom right column for the experimentally tested specimen (figure 7c). The same limit state pattern was predicted numerically as shown in figure 7d. For the masonry infilled panel with one opening in specimen CU2 the sequence of limit states included successively (figure 7e) a) Infill to frame separation b) diagonal cracks of the infill at the window corners c) shear crack in top left column , and d) diagonal crack in the right bottom masonry pier and shear crack mid-height of the right column. The same limit state pattern was predicted numerically (figure 7f).

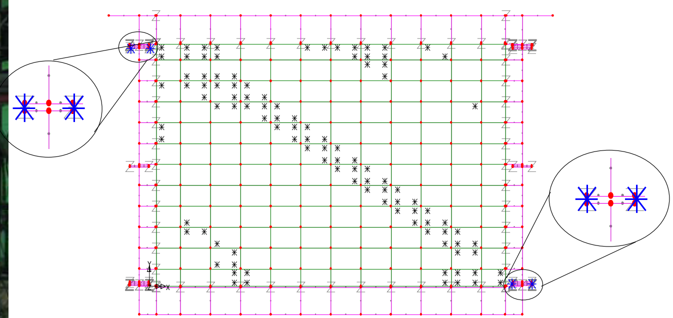




7a) Damage pattern [8] 7b) Limit state numerical simulation (denored with *)

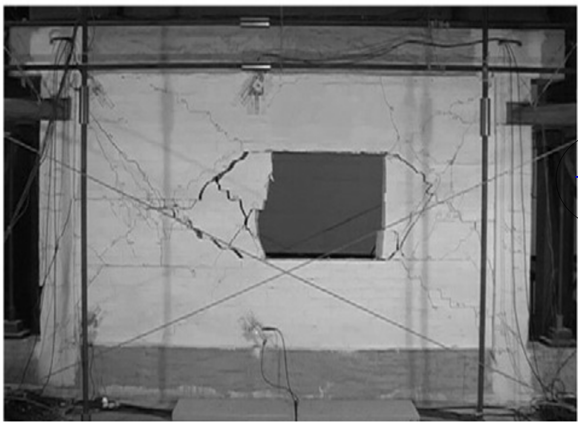
Figures 7a and 7b) Damage pattern of masonry infill observed experimentally for infilled frame F8NR, b) Damage pattern(x) predicted numerically

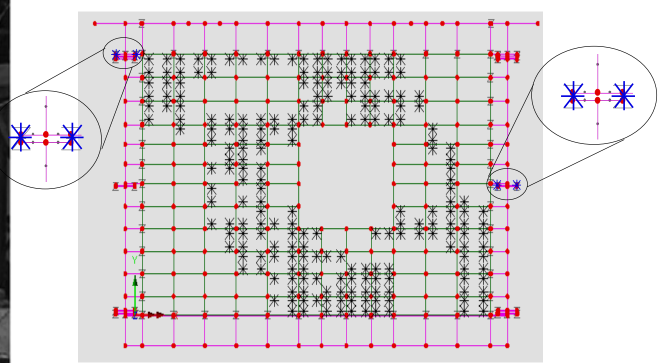




7c) Damage pattern [9] 7d) Limit state numerical simulation (denored with *)

Figures 7c and 7d) Damage pattern of masonry infill observed experimentally for infilled frame CU1, b) Damage pattern(x) predicted numerically





7e) Damage pattern [9] 7f) Limit state numerical simulation (denored with *)

Figure 7e and 7f) Damage pattern of masonry infill observed experimentally for infilled frame CU2, b) Damage pattern(x) predicted numerically

3. VOLVI 6-STOREY MODEL STRUCTURE

The numerical investigation briefly reported in section 2 demonstrates that the proposed numerical model can successfully simulate a number of non-linear mechanisms that can be developed in a single storey masonry infilled frame. In this section it is shown that all these non-linear mechanisms (see introduction a, b and c) can be used to simulate the behaviour of a 6-storey masonry infilled R/C frame structure. This 6-storey masonry infilled R/C frame model structure was constructed and tested at the European Test Site at Volvi, by Manos et al [13], [14], [15]. This structure was utilized for the verification of the proposed numerical simulation.The investigation included two types of analyses; an initial linear dynamic analysis to predict numerically the recorded dynamic response of this structure followed by a non-linear “push over” step-by-step type of numerical analysis of the same infilled frame structure. It must be pointed out that both types of numerical simulations make use of only in-plane stiffness and strength behaviour of the masonry infills units whereas the out-of-plane behaviour and its possible effects, although important, are not addressed here.

3.1. Description of the model structure

This model building was constructed and instrumented at the European Test Site of Volvi in order to monitor its dynamic response under prototype earthquake conditions. An extensive sequence of low-amplitude dynamic tests has been performed. The model included one-bay in each direction, 6-storey, of 1/3-scale with overall external dimensions 1810mm (length) x 5000mm (height) in the weak direction and 1910mm(length) x 5000mm (height) in the strong direction. Its floor level had a height of 1000mm. The cross-section of the columns was 110mmx110mm and that of the beam 100mmx155mmm. In figures 8 a, b and c the masonry infilled, R/C , 6-storey structure is depicted. The basic properties for the concrete and the reinforcement have been monitored through samples taken during construction. Different structural configurations of the 6-storey structure have been tested at the European Test Site of Volvi, in the period of its existence (Manos at al [13], [14], [15]). The structural configuration that is utilized here includes apart from the existing added weight of 50KN, an extra weight of approximately 30KN on the top of a sixth floor specially constructed for this purpose (Masonry scheme 2b, September 1997 – January 2004).

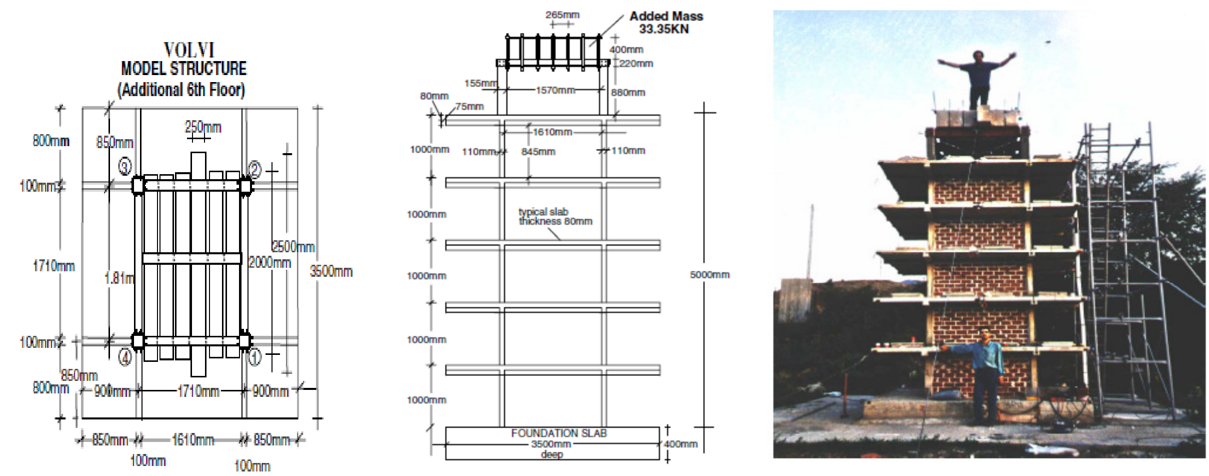


Figure 8 a)Plan view of 6-storey masonry infilled frame structure in European test site at Volvi

b)Side view of 6-storey “bare” frame structure in European test site at Volvi

c) Side view of 6-storey masonry infilled frame structure in European test site at Volvi

Brief information on the masonry infill used in this model building is listed in table 5. Relevant masonry properties and additional information are given by Thauampteh [2].

Description of the 6-storey structure masonry infill	Masonry Infill thickness (mm)	Technical description of the interface between frame and infill	Longitudinal reinforcement ratio (ρ)
Initial unreinforced Masonry with mortar type **V1**	58.5	mortar **V1** thickness **10mm** (without plaster)	1.01%

Table 5: Outline of all specimens for the 1st and 2nd groups of specimens

As already mentioned, the influence exerted by the interface between the masonry infill and the surrounding frame was also examined in both studies by Thauampteh [2] and by Soulis[1]. Tables 6, 7 and 8 list the mechanical properties of the materials used in the construction of the specimens. The masonry infill panel was connected to the surrounding frame with weak mortar (type V1) similar to the one used in the construction of F2N specimen Thauampteh [2]. In this numerical analysis the influence of this interface is examined together with three different cases of foundation-soil stiffness conditions.

Masonry infill	Masonry Infill thickness (mm)	Compres. strength of masonry (N/mm^2)	Shear strength of masonry diagonal compression (N/mm^2)	Compres. strength of masonry units (N/mm^2)	Compres. strength of concrete columns (N/mm^2)	Compres. strength of concrete slabs (N/mm^2)	Compres. strength of mortar cylinders (N/mm^2)
V1	58,5	2,765	0.180	6.50	26	15,6	1.125

Table 6**:** Strengths of masonry infills and concrete used in the specimens [10]

A/α	Yield stress f_{sy} (N/mm^2)	Ultimate strength f_{su} (N/mm^2)	Strain at yield ε_{sy} (%)	Strain at ultimate stress ε_{su} (%)	Young Modulus (N/mm^2)
Φ5.5	338	425	0.8	22.0	$6.5X10^4$
Φ5.5 stirrups	319	542	0.6	20.0	$6.5X10^4$

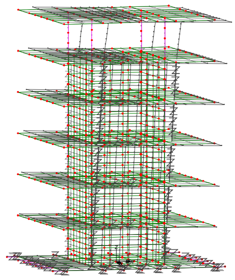
Table 7**:** Tensile strength of the reinforcement used in the specimens [10]

A/a	Simulation of joint interface between frame and infill	**E** Young Modulus (N/mm^2)	**G** Shear Modulus (N/mm^2)	$\mathbf{f_k}$ Measured Compressive Strength of mortar (N/mm^2)	$\mathbf{f_{tn}}$ Assumed Tensile Strength of mortar (N/mm^2) (as % of f_c)	**τo** Local bond shear strength of mortar (N/mm^2)	**μ** friction coefficient
1	**V1** mortar	150	65	1,20	0.12(10%)	0.157	0.20

Table 8**:** Mechanical properties of the mortar joint located between the infill and the surrounding frame (mortar type V1)

3.2. Three dimensional elastic numerical simulation of the dynamic response of the 6-storey model structure

Initially, the results from in-situ tests are presented here dealing with the dynamic response of a 6-storey reinforced concrete (R/C) frame structure with masonry infills, which was built for this purpose at the Volvi European Test site. Summary results from the measured main eigen-frequency values resulting from simple pull-out tests on the 6-storey specimen, with masonry infills in all stories, with steel diagonals and extra mass are included in Table 9. A numerical simulation of the 6-storey (R/C) frame structure with masonry infills utilizing the all the provisions considered in section 2 was built and a dynamic analysis was performed. The numerical simulation included a 3-D simulation of the foundation slab, of all the concrete members (slabs, beams, columns), of the masonry infills, the interface between the masonry infill and the surrounding frame and finally the 6th storey extension. The simulation of the foundation flexibility was approximated by introducing linear springs at the foundation-soil interface. Three different numerical analyses were performed assuming three different levels of soil-foundation stiffness: a) Infinite stiffness (base fixity), b) Medium stiffness (flexible base with foundation-soil interface of E=200Mpa), c) Low stiffness (flexible base with foundation-soil interface of E=10Mpa). Table 9, includes the corresponding summary eigen-frequency results of the numerical simulation studies that were performed assuming these three different soil-foundation stiffness types.

Description of the Structural Configurations for the Volvi Model Structure	1st x-x (Hz) Translational Eigen-frequency	1st y-y (Hz) Translational Eigen-frequency	1st φ (Hz) Torsional	Measuring Procedure
Masonry scheme 2b, June 1998. 6th storey extension Infills in all 5 lower stories (with diagonals) . Measured values [14]	5.005*	4.761*		Permanent & Portable Instruments
Fixed base Values of eigen-frequencies numerically predicted	5.40**	5.31**	5.48**	
Flexible base E=200Mpa Values of eigen-frequencies numerically predicted	4.75**	4.67**	5.12**	
Flexible base E=10Mpa Values of eigen-frequencies numerically predicted	2.58**	2.45**	2.69**	3-d Numerical simulation

* Experimental value., ** Numerical simulation results

Table 9: In-situ measured and numerically predicted vibration frequencies

3.3. Push over non-linear analysis of the masonry infilled 6-storey model structure

For this non-linear "pushover" analysis of the 6-storey structure, the maximum target displacement at the top was set equal to 61mm, with the displacement profile along the height of the structure assumed to be triangular. The in-plane displacements resulting in this way were imposed at each floor level in a gradual increasing step-by-step fashion. The numerical simulation discussed in section 2 for the R/C masonry infilled frames is applied this time for all the 16 masonry infilled bays that compose this 6-storey masonry infilled frame structure (figure 8c). The horizontal load-horizontal displacement "push-over" response curve resulting from this simulation, which as mentioned includes all non-linear mechanisms for the masonry infill, the surrounding R/C frame, the interface between the masonry infill and frame, is depicted in figure 9. The simulation of the masonry infill, was done utilizing non-linear thick shell finite elements. The isotropic nonlinear material law of Modified Von Mises was utilized for this purpose. The adopted mechanical properties of the initial and the strengthened masonry infill panel are listed in Table 10.

Young Modulus (N/mm^2)	1000
Poisson Ratio	0,20
Tensile strength (N/mm^2)	0,30
Compressive strength (N/mm^2)	1,2
Softening Modulus under compression (N/mm^2)	-10
Softening Modulus under tension(N/mm^2)	-10

Table 10: Mechanical properties used for the description of Von Mises failure criterion of the masonry infills

The numerical simulation of the interface between the prototype R/C frame and the masonry infill is done in the same way as described in section 2.1, utilizing non-linear joint elements in the axial and transverse direction. This interface is assumed to have been built with mortar type V1 (see Thauampteh 2] and Table 8). The simulation of the surrounding R/C frame was also performed utilizing the same simulation technique as described in section 2.1.

3.4 "Pushover" curves and Predicted damage for the numerical simulation of the 6-storey frame of Volvi

Figure 9 depicts the base shear – top horizontal displacement curve obtained for the fully non-linear "pushover" analysis for the 6-storey masonry infilled frame structure assuming base fixity. In the same figure the base shear – top horizontal displacement curve is also obtained assuming a flexible foundation with a foundation-soil interface of E=10Mpa. The target top displacement corresponds to approximately 1% angular deformation of the 6-storey masonry infilled frame structure. Figures 10a,b, and 11a,b depict in more detail the predicted damage pattern (shown with **x** signs) of the masonry infills of the 6-storey masonry infilled structural formation resulting from this "push over" type of analysis for two levels of angular deformation a) 0,41%, b)0,84%.

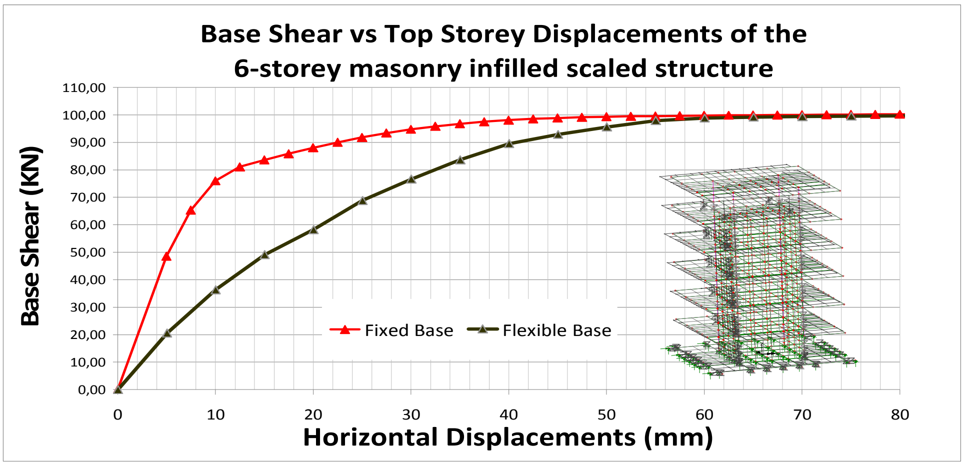
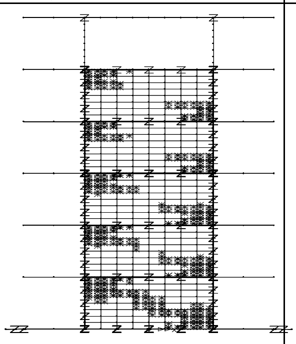
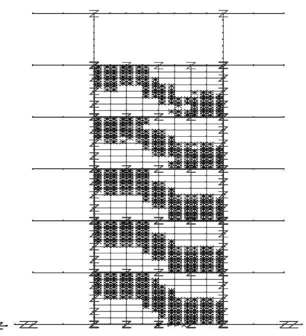
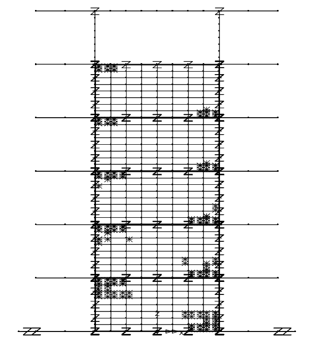
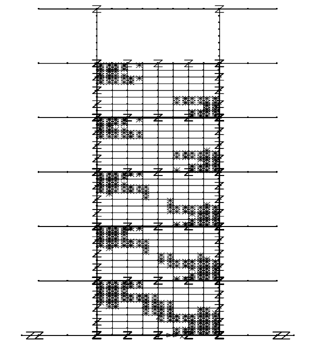


Figure 9: Comparison between push over curves obtained for different foundation conditions

γ=0,0042 %₀(dx=25mm)	γ=0,0084 %₀(dx=50mm)	γ=0,0042 %₀(dx=25mm)	γ=0,0084 %₀(dx=50mm)
			
a) Fixed Foundation	b) Fixed Foundation	a) flexible foundation	b) flexible foundation
Figure 10: Masonry infill damage patterns, as predicted for top storey target displacement equal to a) 4,1‰, b)8,4‰ of the building height with fixed foundation		Figure 11: Detail of masonry infill damage patterns, as predicted for top storey target displacement equal to a)4,1‰, b)8,4‰ of the building height with flexible foundation	

As can be seen in figure 9, the level of the soil-foundation stiffness influences significantly the non-linear “push-over” response of this 6-storey masonry infilled frame structure. The flexibility of the foundation decreases the shear deformation of the masonry infills and the predicted damage levels (figures 11a and 11b) more than for the case of rigid foundation (figures 10a and 10b). Instead, the predicted damage patterns of the masonry infills in this structure, assuming base fixity, develops as expected at a much lower amplitude of top horizontal displacement. The limit state pattern of the masonry infill predicted in this case is along its diagonal (figure 10b). In this sense the flexibility of the foundation acts as a protective mechanism, provided it is designed to be able to sustain such deformations without any side-effects. Moreover, as concluded by an additional experimental sequence conducted with another model structure at the Volvi European Test Site [15] soil-foundation-structure interaction could lead non-embedded foundations to non-linear behaviour resulting in uplift and/or soil plastification. Therefore, such non-linear foundation responses if controlled by design could be a beneficial source of energy absorption. Otherwise, they may lead to excessive displacements of the superstructure and to foundation failure.

4 CONCLUDING OBSERVATIONS

1. The strength and the monotonic load-displacement behaviour observed during the experiments of single-storey one-bay masonry-infilled R/C frames examined in this study is successfully predicted by the proposed numerical simulation.
2. The proposed numerical simulation represents in a reasonable way the most important influences that the interface between masonry infill and the surrounding frame could exert on the monotonic behaviour of such structural assemblies in terms of stiffness, strength, modes of failure, as demonstrated from the observed behaviour.
3. The development of a shear limit state at the predetermined positions of columns of the surrounding R/C frame observed during the experiments as well as the damage patterns for the masonry infill, in terms of crack propagation are also successfully predicted.
4. This numerical simulation was next validated by a 3-D numerical model for a multi-storey masonry infilled frame model structure located at the Volvi European Test Site. Good agreement could be seen between measured eigen-frequency values and predictions from linear dynamic analysis performed assuming different foundation conditions.
5. Finally, this numerical simulation was successfully applied to the same multi-storey masonry infilled frame model structure located at the Volvi European Test Site this time employing a non-linear “push over” type of analyses that included all the non-linear mechanism employed before. From the obtained results it can be demonstrated that the flexibility of the foundation decreases the shear deformation of the masonry infills and the predicted damage levels more than for the case of rigid foundation. In this sense the flexibility of the foundation acts as a protective mechanism, provided it is designed to be able to sustain such deformations without any side-effects.

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