INFLUENCE OF MASONRY INFILLS ON THE EARTHQUAKE RESPONSE OF MULTI-STORY REINFORCED CONCRETE STRUCTURES

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SUMMARY

The most important findings are included here from an experimental investigation on the behavior of two R.C. frame model structures with masonry infills subjected to a variety of horizontal base motions that included simulated earthquake excitations. Variations of the fundamental dynamic characteristics of these test structures, influenced from the sequence of building the infills as well as of the progressive intensity of the ground motion, are presented and discussed. The first structure is a 7-story 1/12.5 scaled 2-D planar frame model, which was tested at the Earthquake Simulator whereas the second, a much larger structure, is a 6-story 1/3 3-D frame model located at the European Test Site at Volvi. Both structures were examined with and without masonry infills. Through the recorded results, together with predictions from numerical simulations, this paper discusses influences arising from the incorporation of the masonry infills in various configurations.

INTRODUCTION

In what follows two test sequences will be presented; first the 7-story 2-D planar frame model, which was tested at the Earthquake Simulator of Aristotle University and next the 6-story 3-D frame model located and tested at the European Test Site at Volvi.

THE 7-STORY PLANAR FRAME MODEL

This structure is a 1:12.5 small-scale model of a 7-story one-bay framed prototype, although such a structure has not been tested elsewhere in larger dimensions. Small-scale model techniques for both R.C. and masonry structural elements have been validated during an extensive long term investigation conducted at the Earthquake Simulator Facility of Aristotle University. Successful simulation of the cyclic behavior of single-story one-bay R.C. infill frame in small scale has been reported before (Manos 1993).

Basic Characteristics of this testing sequence:

The basic dimensions in elevation as well as the extra mass in amplitude and distribution along the height of this model structure are depicted in figures 1a and 1b; reinforcing details are also depicted in figure 1c. The compressive concrete strength was 14Mpa (1.96ksi); the yield stress for the longitudinal reinforcement was 323Mpa (45ksi). Model brick masonry infill panels, shown in figure 1b, were incorporated at specific stages of the test sequence, as will be explained. The ultimate shear stress of these panels, as found from diagonal tension tests was equal to 0.18Mpa (256psi). The out-of-plane response was restrained during testing. The sequence of tests included dynamic excitations as well as simulated earthquake tests (figure 2). The 1st series of simulated earthquakes included three tests; e.g. Taft-0.06 (Test No. 8, low-intensity), Taft-0.6 (Test No. 9, low-to-moderate intensity) and Taft- 2.0 (Test No. 10, moderate to high intensity). During this last test the masonry infill at the 1st story developed clear signs of distress in the form of horizontal cracking. This masonry infill was demolished and a new masonry panel was rebuilt before continuing with the testing sequence. The 2nd series of simulated earthquakes included two tests; Taft-0.3 (Test No 16) was again of low intensity whereas Taft-4.0 (Test No. 17) moderate to high intensity. During this last test the masonry infill at the 1st story again developed clear signs of distress, this time in the form of diagonal cracking. The testing sequence included a 3rd series of

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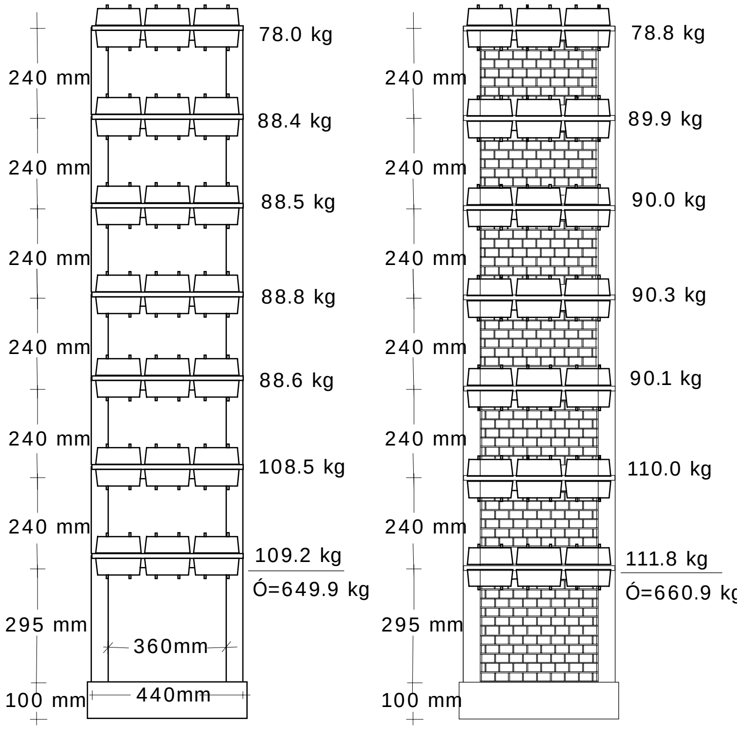
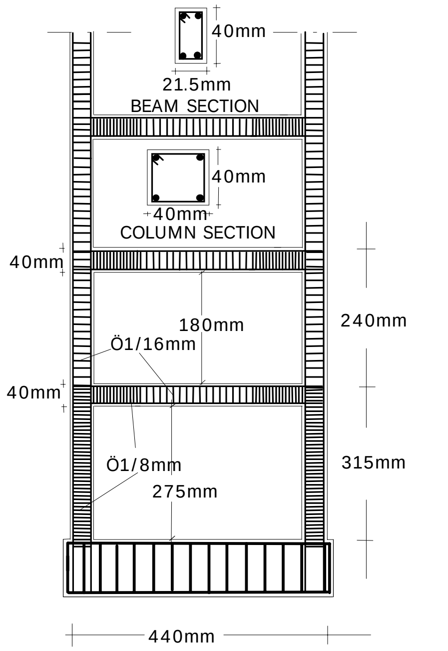
simulated earthquake tests (No. 20 and 21). These tests were based on a ground motion recorded at Edessa during the Griva, 1990 earthquake in Greece.





Figures 1a, b 7-story "frame" model without and with masonry infills

Figure 1c. "Frame" Reinforcing details

During this prototype earthquake a 6-story building suffered extensive damage to its masonry infills. The numerical investigation of the earthquake response of this structure according to the provisions of the current 1992 Greek Seismic Code could not predict sucessfully the observed damage. The cost of repairs represented a considerable percentage of the total building cost for an event that was well below the maximum design earthquake. The severity of the simulated earthquakes used in the laboratory can be seen through the response spectra curves, depicted in figure 3, for Test No. 10 (Taft Span 2.0) and Test No. 17 (Taft Span 4) for the same damping ratio equal to either 5% or 10% of critical.

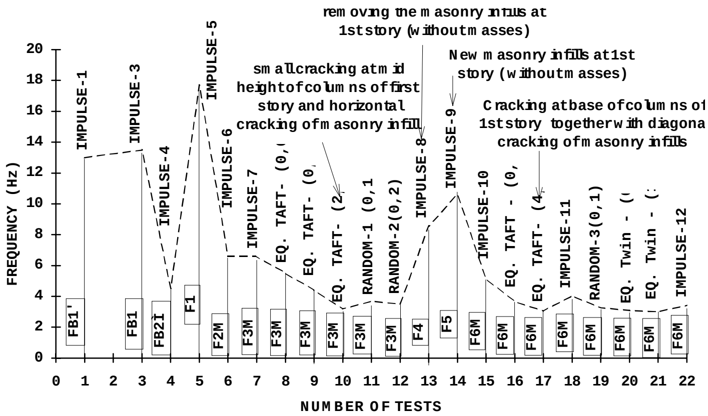


Figure 2 7-story frame Sequence of Tests

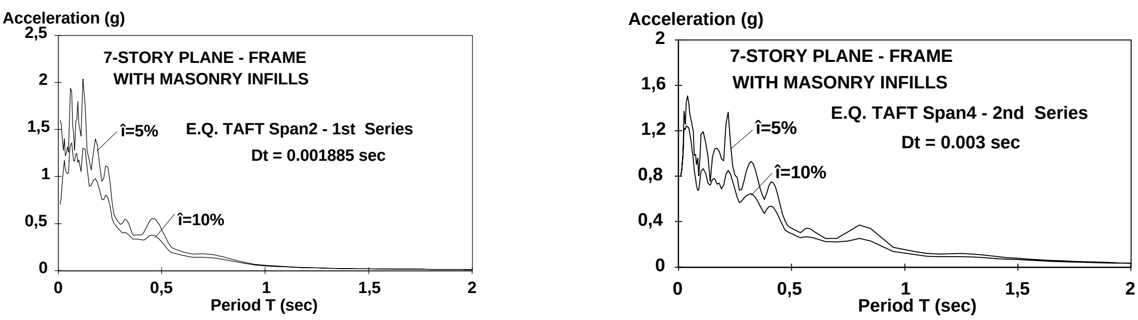


Figure 3a and 3b sponse Spectra of the Simulated Earthqakes Used in the Laboratory

Table 1 The various "frame" structural configurations

Frame **FB1** Bare frame without extra mass	Frame **F1** Infills in all but 1st Story without extra mass	Frame **F4** Infills in all but 1st Story without extra mass
Frame **FB2M** Bare frame with extra mass	Frame **F2M** Infills in all but 1st Story with extra mass	Frame **F5** Infills in all Stories without extra mass
	Frame **F3M** Infills in all Stories with extra mass	Frame **F6M** Infills in all Stories with extra mass

Table 1 lists the various structural configurations that resulted when this frame model was combined with the positioning of the artificial mass and the construction sequence of the masonry infills. The sequence of testing was FB1 - FB2M - F1 - F2M - F3M - F4 - F5 - F6M (figure 2). Frame F4 resulted from F3M by demolishing the masonry infill of the 1st story whereas frame F5 resulted from F4 by removing the extra mass and rebuilding a new masonry infill at the 1st story.

Results of Dynamic Tests and Discussion of the Observed Response:

The objective here was to examine the variation of the fundamental dynamic parameters during the testing sequence.

The variation of the fundamental translational frequency is depicted in figure 2. The value of this frequency was 13Hz at the beginning of the test sequence, when the model corresponded to a completely un-cracked virgin structure without masonry infills or extra mass (Frame FB1). When the mass was added, it became 4.52Hz (Frame FB2M). Next, when the extra mass was removed and the masonry infills were built in all but the 1st story, they introduced a considerable increase in the stiffness (fundamental frequency 17.7Hz, Frame F1 and 6.59Hz when the extra mass was added for Frame F2M). At this stage the construction of the 1st story infill did not show any stiffness increase.

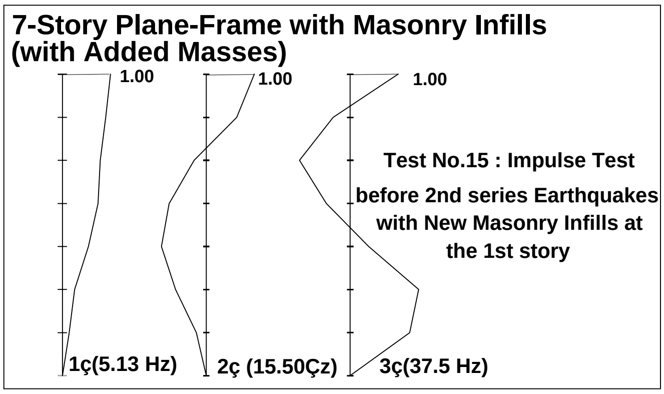


Figure 4 Test No. 15 F6M

This must be due to the low level of dynamic excitations used to assess the dynamic structural properties. Due to the damage that was inflicted to the masonry panels, particularly that of the 1st story, as well as to mostly non-visible micro-cracking of the R.C. columns, there was a considerable drop in the stiffness of the structure. This can be observed at the end of the 1st series of simulated earthquake tests (fundamental frequency 3.5Hz, Frame F3M). Some of the stiffness was regained by demolishing the damaged masonry infill and building a new one in the sequence that was described above (fundamental frequency 5.13Hz, Frame F6M). The mode shapes prior to the commencement of testing for this sequence are depicted in figure 4. During the 2nd series of simulated earthquake tests a significant loss of stiffness was also observed (fundamental frequency 3.26Hz, Frame F6M)

Simulated Earthquake Test Results and Discussion of the Observed Response

Test No 10, Taft, span 2.0, 1st series frame F3M: The response is maximized for a frequency value equal to 3.18Hz. The base shear and overturning moment for this test have maximum values equal to 0.288t (634lb) and 0.374tm (2703lbft), respectively. The equivalent stiffness of the frame as the ratio of the base shear over the 1st story drift is equal to 185t/m (124kips/ft).

Test No 17, Taft, span 4.0, 1st series Frame F6M : Maximum response is attained at 3.05Hz. The maximum diagonal displacement for test No. 17 is3.725mm (0.147in), nearly twice as much as for test No. 10 (1.893mm-0.075in). The overturning moment and base shear for this test are shown in figures 15a and 15b together with the 1st story drift. Maximum base shear and overturning moment values equal to 0.305t (672lb) and 0.361tm (2608lbft), respectively. The equivalent stiffness of the frame during this test as the ratio of the base shear over the 1st story drift is initially equal to235t/m (158kips/ft). This stiffness value is larger than the corresponding value during test No. 10. However, the rate of stiffness degradation for test No 17 is also larger than that of test No 10, due to the damage in the form of diagonal cracks at the mortar joints of the masonry infills at the base of the structure.

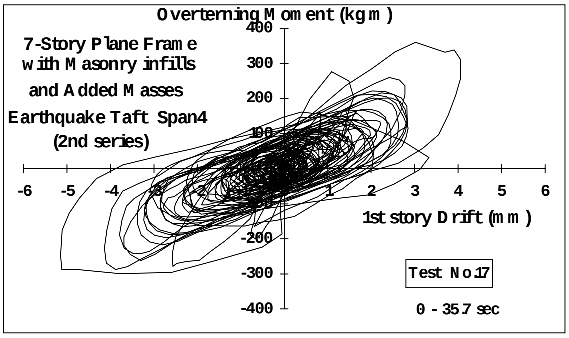


Figure 5a. Test No. 17 Overturning Moment

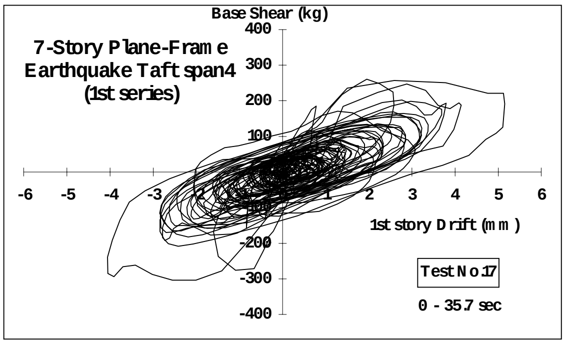


Figure 5b. Test No. 17 Base Shear

Remarks on the response of the 7-story Planar Frame Model

- From the low amplitude dynamic excitation tests the damping ratio for the frame structure was found to be in the range of 5.6% to 6.05% of critical. The initial stiffness of the frame model was 235t/m (158kips/ft); however, a large part of this stiffness was due to the presence of the masonry infills and degraded rapidly, due to the diagonal cracking of the infill, first to values of the order of 100t/m (67kips/ft) and then, towards the end of the response, to values of the order of 20% of the initial stiffness. This fact displays a significant aspect in the behavior of weak masonry infills; for moderate earthquake loads they may retain their stiffness and thus participate up to a degree in the load bearing capacity; however, due to their brittle behavior this participation soon ceases to exist after they are damaged, usually in the form of diagonal cracks.

- In terms of story drift, base shear and overturning moment the model structure developed a satisfactory post-elastic response. It must be born in mind that the frame during test No. 17 developed only slight structural damage (cracking of columns) and its main damage was “non-structural damage” (masonry infills). This damage to the infills corresponded to an equivalent shear strain of the order of 0.005 to 0.007.

- From the presentation of the results of the 7-story frame model structure it can be seen that it developed realistic global response behavior, similar to that of prototype structures. Moreover, the observed damage patterns bear resemblance to observed performance of corresponding prototype structures. As concluded by other researchers, despite the difficulties and limitations involved in the small-scale modeling of the earthquake response of R.C. structures, such models can be useful tools in understanding the complex earthquake behavior of structures.

THE MULTI-STORY MODEL STRUCTURE AT THE VOLVI EUROSEIS-TEST SITE

Summary results from in-situ tests are presented here dealing with the dynamic response of a multy-story reinforced concrete (R.C.) building with and without masonry infills, which was built for this purpose at the Volvi European Test site. The test site is located at the Mygdonian valley near Thessaloniki, Greece, in an area of high seismicity. At the central part of the valley, a number of accelerographs have been concentrated both at the surface as well as at a certain depth in the alluvium soil layer and it is here that a multy-story reinforced concrete structure with masonry infills has been built. This model building is of the weak-column type (Okada, 1992); it was constructed and instrumented at this site in order to monitor its dynamic response under prototype earthquake conditions. Despite the disadvantage of being unable to produce in-situ significant levels of ground motion, when desired, as can be generated by an earthquake simulator at the laboratory, the advantage here is the presence of realistic conditions for both the foundation support and of course the earthquake ground motion. Already, one earthquake, of moderate intensity, subjected the model structure to seismic loads and excited the permanent instrumentation system. Moreover, an extensive sequence of low-amplitude dynamic tests have been performed with some of the results presented in summary form in this paper, together with predictions from numerical simulations as will be outlined in the following. Figures 6a and 6b depict the dimensions of this structure that is made of reinforced concrete cast in-situ. The basic properties for the concrete and the reinforcement have been monitored through samples taken during construction. The average concrete strength was found equal to 26Mpa for the columns and 15.8Mpa for the slabs; the yield stress of the longitudinal and transverse reinforcement was found equal to 338Mpa and 319 Mpa, respectively. The masses at each story, both from the dead weight of the structural elements as well the extra mass that was symmetrically distributed at each story slab are recorded in detail (Manos 1995).

Studied Structural Configurations:

This model structure, at the European Test Site at Volvi, must be considered in the 5 year period of its existence in the following five basic structural configurations .

a. 5-story reinforced concrete structure without added weight and without any masonry infills (“Virgin” structure, September - November, 1994).

b. 5-story reinforced concrete structure with 5 tons added weight but without any masonry infills (“Bare” structure, November 1994 - June, 1995).

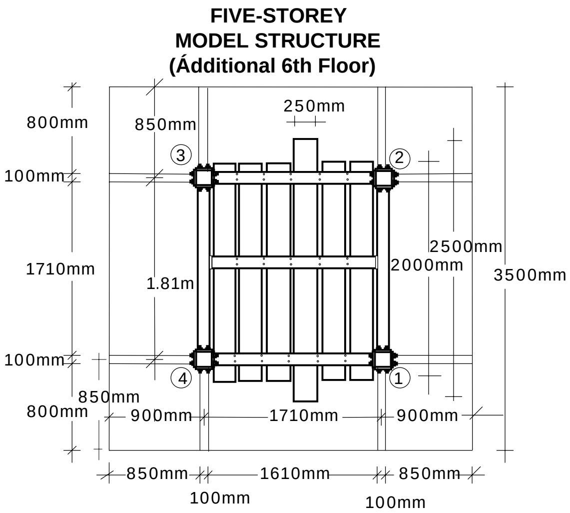
c. 5-story reinforced concrete structure with 5 tons added weight and with masonry infills in all but the ground floor (Masonry scheme 1, July 1995 - January 1997).

d. 5-story reinforced concrete structure with 5 tons added weight and with masonry infills in all floors (Masonry scheme 2, February 1997 – September 1997).

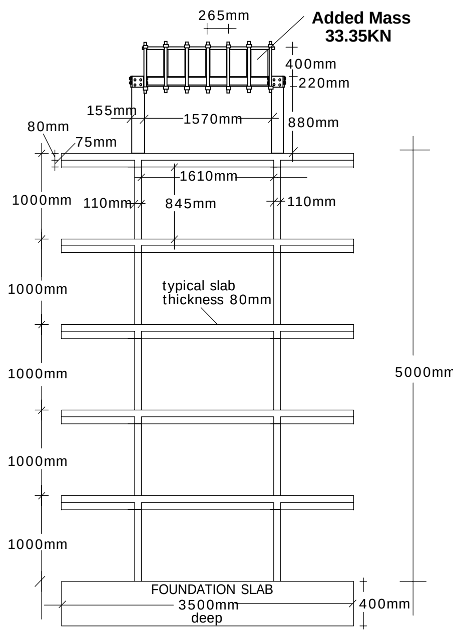
e. 6-story reinforced concrete structure with masonry infills in all the five lower floors and 8.5 tons extra mass (3.5tons on the 6th story extension, September 1997 – today).

The above five basic configurations were combined at times with the selected presence of a number of diagonal steel cables at the bays of the story frames to thus form various sub-formations. The first main sub-formation is when all diagonal steel cables are active (by being pre-stressed) and the second main sub-formation when all diagonals are inactive (being loose). These sub-formations were employed in all three configurations, i.e. the “Virgin”, the “Bare” and the “Masonry scheme 1” and “Masonry scheme 2” In all these cases the symmetry in the mass and stiffness distribution is maintained with respect to both x-x and y-y axes. In addition, for the “Bare” structure with added weight and without masonry infills an additional asymmetric scheme of active diagonals was employed, whereby the presence of the diagonal cables was non-symmetric. Moreover, for the fifth structural configuration that is the structure with a 6th story extension, the steel diagonal cables are always present and pre-stress at the all the bays of the 6th story.

PLAN



Cross Section



Figures 6a and 6b. Reinforced concrete structure with 5tons (11kips) added weight and with masonry infills in all floors (Masonry scheme 2, January 1997 - today).

Instrumentation - Data Analysis of the Measured Dynamic Response from simple Pull-out Tests:

A permanent instrumentation system was utilized to monitor the earthquake structural response. This system, constructed and then proof-tested at Aristotle University of Thessaloniki, is operating at the test site on a continuous basis. Summary results of this data analysis from a large number of simple pull-out tests are included in Table 2. When viewing these results careful consideration must also be paid at the structural changes introduced to the test structure during this period, as outlined in the previous paragraph. This table also includes the corresponding summary results of the numerical simulation studies that were also performed. A special technique was developed for deriving the dynamic characteristics of this model structure. By combining the large volume of the response measurement data from the various in-situ low-vibration sequences the most important mode shapes and eigen-frequencies were identified.

Table 2. Summary of measured and predicted eigen-frequencies for the Volvi test structure.

Description of the Structural Configurations for the Volvi Model Structure	1st x-x (Hz) Translational	1st y-y (Hz) Translational	1st φ (Hz) Torsional	Measuring Procedure
Without diagonals, without added mass “Virgin” structure, September 1994	2.875 {2.852}**	2.875 {2.846}**	2.75 {2.774}**	Real-time Analyzer
Without diagonals, with added mass “Bare” structure, November 1994	2.375 {2.413}**	2.375 {2.408}**	2.50 {2.373}**	Real-time Analyzer
With diagonals, with added mass “Bare” structure, November 1994	2.625 {2.640}**	2.625 {2.640}**	2.50 {2.711}**	Real-time Analyzer
Without diagonals, with added mass “Bare” structure, May 1995	2.375m {2.478}**	(2.375)* {2.473}**	2.375 {2.438}**	Real-time Analyzer
Without diagonals, with added mass “Bare” structure, May 1995	2.440 {2.496}**	(2.440)* {2.501}**	2.440 {2.434}**	Permanent Instruments
With diagonals, with added mass “Bare” structure, May 1995	2.563 {2.590}**	(2.563)* {2.588}**	2.563 {2.692}**	Permanent Instruments
Masonry scheme 1, October 1995 with diagonals, with added mass masonry infills in all but ground floor	4.150 {4.127}**	(4.150)* {3.94}**	{4.20}**	Permanent Instruments
Masonry scheme 1, January 1997, infills in all but ground floor, with diagonals, added mass	4.150 {4.127}**	4.270 {4.102}**	4.520 {4.258}**	Permanent Instruments
Masonry scheme 1a, January 1997, added mass, infills in all but ground floor, diagonals only at ground floor	4.150 {4.127}**	{4.102}**	4.520 {4.258}**	Permanent and Portable
Masonry scheme 1b, February 1997 with no diagonals, added mass, masonry infills in all but ground floor	4.03 {4.047 }**	4.03 {4.027}**	4.27 {4.168}**	Permanent Instruments
Masonry scheme 2, February 1997, added mass, masonry infills in all stories (with no diagonals)	4.395 {5.537}**	4.395 {5.453 }**	{6.098}**	Permanent Instruments
Masonry scheme 2, September 1997, added mass and masonry infills in all stories (with diagonals)	5.737 {5.666}** x-φ	6.226 {5.57}**y-φ	{4.883 }**	Permanent Instruments
Masonry scheme 2b, October 1997 6th story extension. Infills in all 5 lower stories (with diagonals)	{4.745}**	4.760 {4.673}**	{5.121}**	Permanent Instruments
Masonry scheme 2b, June 1998. 6th story extension Infills in all 5 lower stories (with diagonals)	5.005 {4.745}**	4.761 {4.673}**	{5.121}**	Permanent and Portable

( )* Assumed experimental value. No y-y translational pull-out was performed { }** Numerical simulation results, see section 2.3.

From October 1995 till January 1997 the 5-story structure was with masonry infills in all but the ground floor (pilotis). Masonry infills were added at the ground floor in February 1997. The last low-intensity vibration sequence before adding the ground floor masonry infills was performed during January 1997. From February 1997 till September 1997 the 5-story structure was with masonry infills in all five stories. A sixth story was added to the structure together with additional mass by the end of September 1997. The last low-intensity vibration sequence before adding the sixth story was performed during 12th September, 1997. Prior to these tests additional low-intensity vibration sequences were performed; one during February 1997 immediately after the construction of the ground floor masonry infills and another one three months later (April 1997). As was to be expected the presence of the masonry infills in the upper stories has influenced significantly both the eigen-frequency values and mode shapes that were obtained from the structural configurations without the masonry infills. The following conclusive observations can be deduced from these results at this stage:

- The fundamental frequency of the structure without the ground floor masonry infills exhibited only relatively small changes with time. When masonry infills were added at the ground floor apart from the large increase in the observed fundamental frequency values a variation of these values with time was also observed, due to the increase of the stiffness of the infills with time. This was occurred within a period of the first eight months.

- The presence of the diagonals in the upper four floors has little effect on the stiffness of the structure, due to the stiffness of the masonry infill. Furthermore, the presence of the diagonals only at the first story (without infills) does not result in any significant increase of the structural stiffness. As expected, the addition of the 6th floor extension with its extra mass lowered the fundamental frequency values.

Table 3	**Predicted Eigen-frequencies (Hz)**						Mass	Remarks
Simulation	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	(t)	
5stry5d.	2.438 φ	2.473 y	2.478 x	7.101 φ	7.268 y	7.283 x	17.85	Higher Young's Modulus
5str5da.	2.434 φ	2.496 x	2.501 y	7.197 φ	7.384 x	7.397 y	17.85	Columns axially stiff
5str5ea.	2.640 y	2.640 x	2.711 φ	7.613 φ	7.800 y	7.804 x	17.85	Diagonal Steel Cables
5str5eb.	2.507 φ	2.57x-y	2.638 φ	7.286 φ	7.60x-y	7.71φ-x	17.85	Asymmetric Diagonals
5str5fa.	2.588 y	2.590 x	2.692 φ	7.563 φ	7.766 y	7.770 x	17.85	Flexible Foundation
5str5ga.	4.102 y-y	4.127 x-x	4.258 φ-φ	14.318 y-y	14.493 x-x	14.841 φ-φ	19.53	Infills in all but ground floor where diagonals. Flexible Foundation
5str5gb.	4.027 y-y	4.047 x-x	4.168 φ-φ	14.200 y-y	14.370 x-x	14.757 φ-φ	19.53	Infills in all but ground floor Without diagonals
5str5ia.	5.453 y-y	5.537 x-x	6.098 φ-φ	17.764 φ-φ	18.058 y-y	18.344 x-x	19.53	Infills in all 5 stories Flexible Foundation
5str5ib.	6.174 φ	6.345 y	6.447 x	17.95 φ	18.61 y	18.91x	19.53	As 5str5ia but with base fixity
5str5ic.	5.570 y-φ	5.666 x-φ	6.355 φ-φ	17.946 φ-φ	18.808 y-φ	19.355 x-φ	19.53	Asymmetric masonry stiffness Flexible Foundation
5str5ja.	4.673 y-y	4.745 x-x	5.121 φ-φ	15.797 φ-φ	16.362 y-y	16.615 x-x	22.90	6th Story extension with extra mass. Flexible Foundation

Numerical Simulations of the Observed Response.

These numerical simulations extended in the linear range only and included a 3-D simulation of the foundation slab of all the concrete parts, of the diagonal cables, of the masonry infills, and finally of the 6th story extension. Provisions were taken within certain of these numerical simulations to incorporate the flexibility of the foundation-soil interface. A brief description of the most significant aspects of each simulation with the summary of the results is given in table 3. Moreover, a comparison between predicted and observed dynamic response properties is included in Table 2.

Measured and Predicted Response During the Earthquake of 4th May, 1995 :

Two earthquakes occurred on the 4th April, and 4th May 1995, with an epicentral distance from the Volvi test site approximately 40Km. Despite the fact that these earthquakes generated relatively low-intensity ground motion at the test-site, The 5-story structure was excited from those two ground motions and the response was recorded and stored by the permanent instrumentation scheme. During this earthquake sequence the 5-story structure at Volvi had the following configuration: the added weights were in place, no masonry infill had been built as yet, and all diagonals were pre-stressed in all floors. The numerical simulation described before was employed and it was successful in predicting the measured earthquake response of this structure.

DISCUSSION OF THE RESULTS - CONCLUSIONS

1. The investigation of the seismic behavior of the 7-story planar frame model structure demonstrated the significant aspect in the behavior of weak masonry infills; for moderate earthquake loads they may retain their stiffness and thus participate up to a degree in the load bearing capacity; however, due to their brittle behavior this participation soon ceases to exist after they are damaged, usually in the form of diagonal cracks. Moreover, it was found that the building sequence influenced the degree that this masonry added stiffness was mobilized. In addition, the 7-story frame model structure developed realistic global response behavior, similar to that of prototype structures. Moreover, the observed damage patterns bear resemblance to observed performance of corresponding prototype structures. Thus, despite the difficulties and limitations involved in the small-scale modeling of the earthquake response of R.C. structures, such models can be useful tools in understanding the complex earthquake behavior of such structures and in verifying the relevant provisions in the seismic codes.

2. Significant variations were observed in the stiffness of the Volvi structure during the various stages of including the masonry infills. The fundamental frequency of the Volvi structure without the ground floor masonry infills exhibited only relatively small changes with time. When masonry infills were added at the ground floor apart from the large increase in the observed fundamental frequency values a variation of these values with time was also observed, due to the increase of the stiffness of the infills with time. This was observed within a period of the first eight months. The presence of the diagonals in the upper four floors has little effect on the stiffness of the structure, due to the stiffness of the masonry infill. Furthermore, the presence of the diagonals only at the first story (without infills) does not result in any significant increase of the structural stiffness. As expected, the addition of the 6th floor extension with its extra mass lowered the fundamental frequency values.

3. The numerical simulation of the dynamic characteristics of the model Volvi building with and without masonry infills was successful. This fact must be attributed to the very effective control of micro-cracking in this structure as well as to the almost exact estimation of the dimensions of the various structural elements and the accurate estimation of the mass of the system. Moreover, a special laboratory investigation was performed to verify the properties of the masonry infill. However, it must be borne in mind that the measurements used and the assumptions employed in the simulations are based on linear-elastic response. The success of the subsequent numerical simulation of the 4th of May, 1995 recorded earthquake response must also be seen in the light of the validity of the linear-elastic response assumptions. Despite this, a small adjustment was necessary in the stiffness and damping derived during the free-vibration in-situ test sequences.

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