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The dynamic response of 5-story R.C. structures in-situ at the European test site at Volvi

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1. INTRODUCTION

Summary results from in-situ tests are presented here dealing with the dynamic response of a 5-story reinforced concrete (R.C.) building, which was built for this purpose at the Volvi European Test site. Only essential information is included, as details on the test site and the 5-story structure are given elsewhere (Pitilakis et.al., Manos et.al.). The test site is located at Volvi near Thessaloniki, Greece, in an area of high seismicity. The 5-story reinforced concrete building 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, a series 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. This work has been partially supported by the European Union, Projects ENV.5V-CT93-0281 and ENV.4-CT.96-0255 (DIR XII).

2. DESCRIPTION OF THE TEST SITE

The Volvi test site is located at the Mygdonian valley, which is bounded from North and South by sloping rocky formations at the villages of Stivos and Profitis, whereas thick layers of alluvia deposits form the central part. Figure 1 shows a transverse cross-section of the valley at its most narrow part; it was selected to deploy a considerable number of accelerographs all along this cross-section in order to record the distribution of the prototype earthquake ground motion along the valley and to identify influences arising from the variation of the stiffness of the various soil-layers. For this purpose some of these instruments are located on the very stiff rocky slopes at the ends of the valley whereas the rest of them were placed at various locations within this section of the valley, as depicted in figure 1. As can be seen, 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. At this central part the permanent facilities of the European test site are also established, and it is here that the 5-story reinforced concrete structure has been built.

3. DESCRIPTION OF THE TEST STRUCTURE

Figures 2a and 2b depict the dimensions of this structure that is made of reinforced concrete cast in-situ. All the material properties for the concrete and the reinforcement have been monitored by tests performed with samples taken during construction. Moreover the masses at each story, both from the dead weight of the structural elements (slab, beams, columns or masonry infills) as well the extra mass that was symmetrically distributed at each story slab are recorded in detail (Manos 1995a,b).

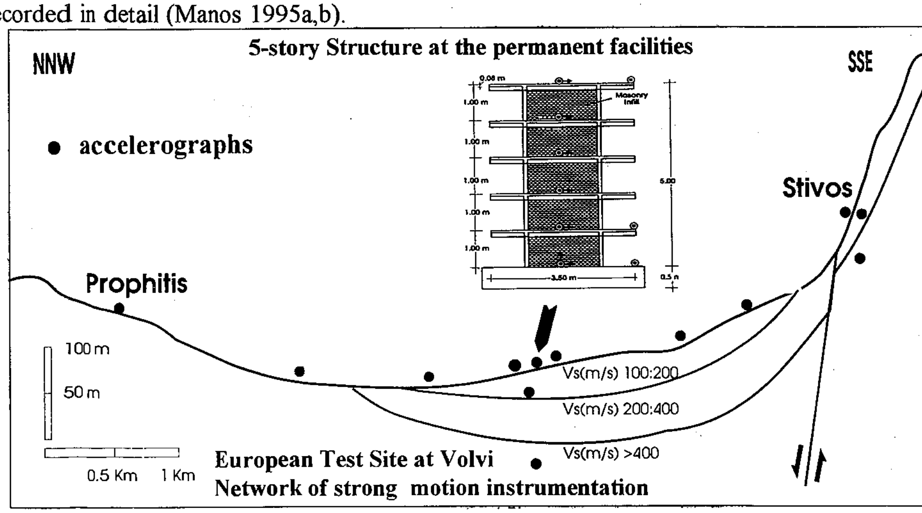
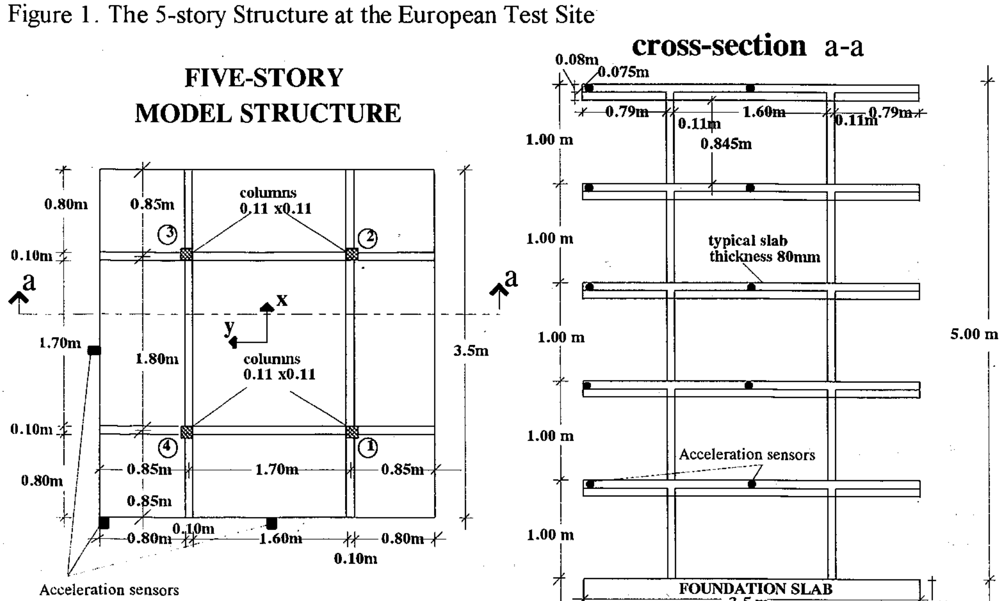


Figure 1. The 5-story Structure at the European Test Site



Figures 2a, 2b Dimensions of the 5-story Reinforced Concrete Framed Structure at Volvi.

Studied Structural Configurations : This 5-story structure, at the European Test Site at Volvi, must be considered in the 30 month period of its existence in the following four basic structural configurations .

a. Reinforced concrete structure without added weight and without any masonry infills ("Virgin" structure, September - November, 1994).

b. Reinforced concrete structure with 5 tons added weight but without any masonry infills ("Bare" structure, November 1994 - June, 1995).
c. Reinforced concrete structure with 5 tons added weight and with masonry infills in all but the ground floor (Masonry scheme 1, July 1995 - December 1996).
d. Reinforced concrete structure with 5 tons added weight and with masonry infills in all floors (Masonry scheme 2, January 1997 - today).

The above four basic configurations were combined 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 as will be explained in section 5.

Table 1. List of low-amplitude dynamic tests conducted in-situ

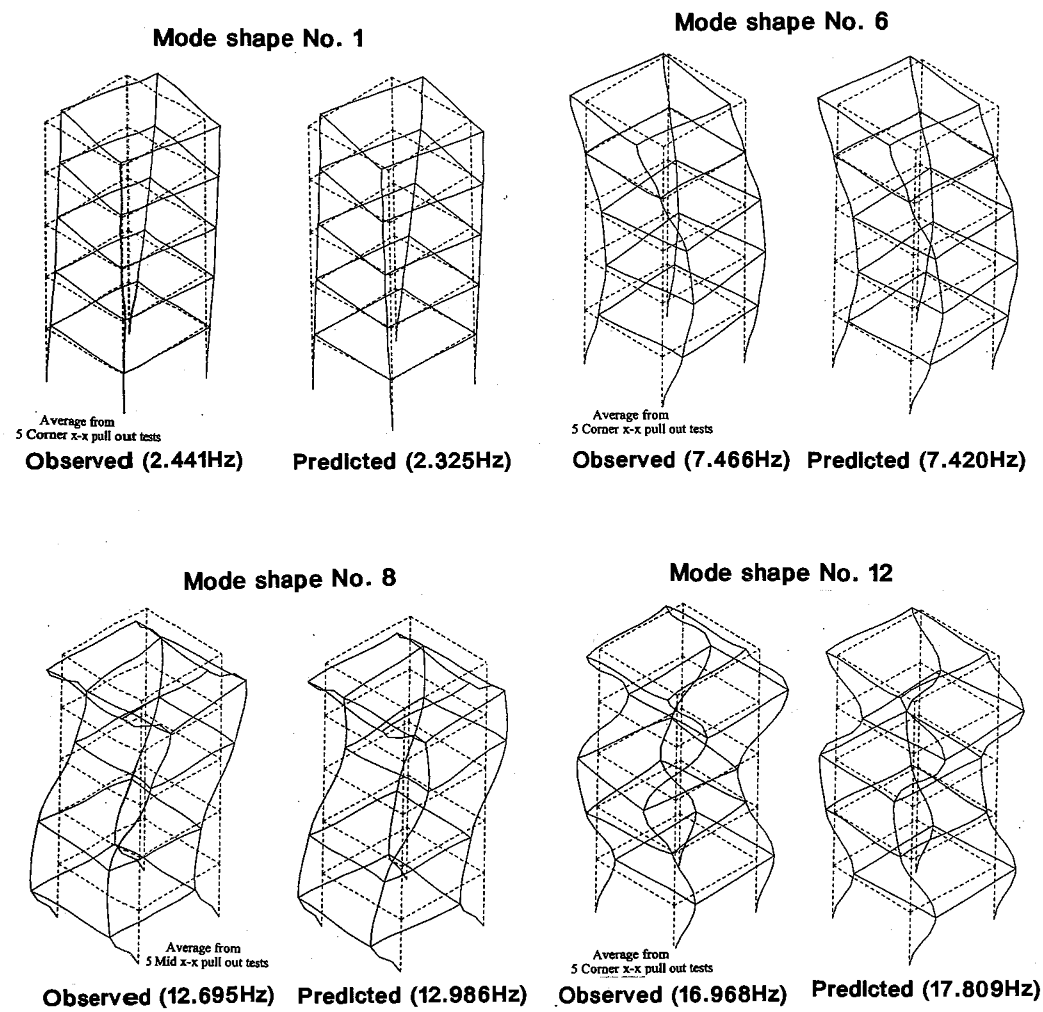
Description of Basic Configuration	Date of testing sequence	Description of tested Sub-Formations	Direction of Pull-out test *	Measuring Procedure
"Virgin" Structure no added weight	Septemb er 1994	without Diagonals	x-x, y-y, φ- φ	Real-time Analyzer
"Bare" Structure added weight, no masonry	Novemb er 1994	without Diagonals all Diagonals (40)	x-x, y-y, φ- φ x-x, y-y, φ- φ	Real-time Analyzer
"Bare" Structure added weight, no masonry	May 1995	without Diagonals with all Diagonals (40) asymmetric Diagonals (20)	x-x, φ- φ x-x, φ- φ	Real-time Analyzer and Permanent Instruments
Masonry Scheme 1 added weight, masonry infills in all but the ground floor	October 1995 January 1997	with all Diagonals (40) with all Diagonals (40)	x-x x-x, y-y, φ- φ	Permanent Instruments

* x-x, y-y are mainly translational pull-outs in the x-x and y-y directions, respectively
φ- φ is mainly torsional pull-out

4. INSTRUMENTATION - LOW AMPLITUDE DYNAMIC TESTS

A permanent instrumentation system was utilized to monitor the earthquake structural response. A total number of sixteen acceleration sensors have been installed at the 5-story building. Each story has three acceleration sensors mounted at the mid-plane of the corresponding slab. An additional sensor is mounted on the foundation block in the axis of symmetry (x-x direction). This 16-channel data acquisition system, constructed and then proof-tested at Aristotle University of Thessaloniki, Earthquake Simulator Facility, is operating on a continuous basis. An extension to the existing permanent instrumentation is planned for the immediate future. This additional instrumentation aims to monitor all aspects of the input ground motion at the foundation level of the test structure. By comparing the recorded ground motion at the free surface by the accelerographs located nearby (at a distance approximately

15m) to those that will be recorded at the foundation, possible variations are hoped to be identified.. Moreover, the permanent instrumentation is combined with portable equipment from the Earthquake Simulator Facility of Aristotle University to conduct low-amplitude dynamic tests. These tests were conducted at various phases of the research program with the structure being at the time at various stage of structural configuration, as explained in section 3. This sequence of tests is listed in Table 1 whereas summary results are presented in Table 2. The analysis of the measured response took place at the Laboratory of Strength of Materials of Aristotle University in order to accomplish the modal identification process through specialized software.



Figures 3. Correlation of modal Characteristics
“Bare” Structure with Asymmetric Diagonal Cables

5. MODAL ANALYSIS OF THE ASYMMETRIC STRUCTURAL CONFIGURATION

A special study was conducted employing the asymmetric scheme for the active diagonal steel cables. The modal analysis in this case was more complex as the modes, to be extracted from the measured response, were coupled, containing translational as well as torsional response. The pull out force in this case was always applied in a direction parallel to the x-x axis; in the first test sequence (symbolized as x-x) the force was applied at the mid-point of the slab-side between columns C1 and C4 (figures 2a,b) whereas at the second test sequence (symbolized as φ- φ) the force was applied at the corner of the same slab-side (nearest to column C4). Each test sequence contains five tests corresponding to the force being applied at a different story (from the 1st to the 5th. The response, measured in this way, was analyzed at the Earthquake Simulator Facility of Aristotle University, by obtaining the fast Fourier transform (FFT) amplitude and phase spectra. Then, for each test sequence, the following steps were taken in combining the response from all fifteen measuring points and thus extracting the mode shapes:

a. First the frequencies for which the response is maximized are selected. These are selected from the peaks of the individual FFT spectrum. In this way a number of frequencies are selected that, for the studied case, belong to the following five frequency bands (2-3Hz), (7-8Hz), (12-13Hz), (16-17Hz), (20-21Hz).
b. For each test and for one of the above frequencies the point in the structure where the FFT amplitude is maximum is found (reference point in each test); next the response in all fifteen points is normalized.
c. For each test and for one of the above frequencies the phase angles of the responses at the 14 measuring points with respect to the response of the reference point are also considered. Taking into account these phase angles a concurrent normalized distribution of FFT amplitudes at the 15 measuring points is found for a given frequency and pull-out test.
d. By following the above procedure for the same frequency value but for the rest of the four tests in the same sequence, and by averaging the FFT amplitude distributions, obtained in this way, an average normalized mode shape is thus derived.
e. The selection of the most appropriate mode shape in each frequency band is finally performed.
f. Next, the modal amplitudes in the 15 points are combined and transformed to represent a 3-D modal response for each of the 5 story-slabs (see figure 3).

6. NUMERICAL SIMULATIONS OF THE RESPONSE

The measured response during all low-amplitude dynamic tests (Table 1) was utilized to assess the stiffness of the five-story structure at its various stages when it represented the structural configurations mentioned previously. This was done employing 3-D structural analysis numerical simulations for the given geometry of the studied structure and its elements and selecting the appropriate material properties. The agreement, obtained in this way, can be judged by the correlation of the measured and predicted fundamental frequency values, listed in Table 2.

Predicted Dynamic Response Characteristics of the "Bare" Structure with added masses and Diagonal Cables : The 5-story building with added masses and without infills, but with the diagonal cables considered as being effective by small amount of pre-stressing, was numerically simulated, in the way previously described. The material properties for the R.C.

cross-sections were the ones found from correlation studies with the structure in the "bare" configuration without diagonals. The cross-sectional properties of the diagonal cables were adjusted so as to have the first three eigen-frequencies, e.g. 1st translational x-x, 1st translational y-y, and first torsional, with values equal to the ones measured in-situ (see Table 2). The axial stiffness of the diagonal cables was kept the same for all stories (EA=90t). The above adjustments were made based on the measured response during the symmetric cases with all diagonals in place. The same numerical approach was also repeated for the asymmetric case. The comparison between the measured and the numerically predicted mode-shapes and frequencies is depicted in figure 3.

Table 2. Summary of measured and predicted eigen-frequencies for the 5-story structure. In italics { }** are the corresponding values obtained from the numerical simulations.

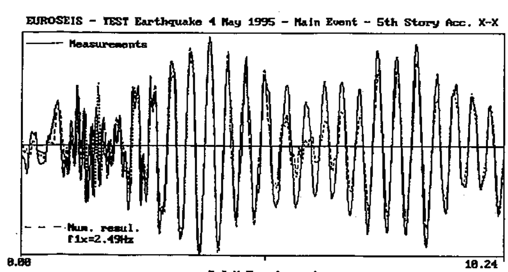
Description of the 5-story Structural Configurations*	1st Translational x-x (Hz)	1st Translational y-y (Hz)	1st Torsional ö (Hz)	Measuring Procedure
without diagonals, without added mass "Virgin" structure, September 1994	2.875 *{2.875}***	2.875 *{2.85}***	2.75 *{2.71}***	Real-time Analyzer
without diagonals, with added mass "Bare" structure, November 1994	2.375 *{2.40}***	2.375 *{2.38}***	2.50 *{2.30}***	Real-time Analyzer
with diagonals, with added mass "Bare" structure, November 1994	2.625 *{2.605}***	2.625 *{2.625}***	2.50 *{2.532}***	Real-time Analyzer
without diagonals, with added mass "Bare" structure, May 1995	2.375 *{2.40}***	(2.375)* *{2.38}***	2.375 *{2.30}***	Real-time Analyzer
without diagonals, with added mass "Bare" structure, May 1995	2.44 *{2.40}***	(2.44)* *{2.38}***	2.44 *{2.30}***	Permanent Instruments
with diagonals, with added mass "Bare" structure, May 1995	2.563 *{2.605}***	(2.563)* *{2.625}***	2.563 *{2.532}***	Permanent Instruments
with diagonals, with added mass masonry infills in all but ground floor Masonry scheme 1, October 1995	4.15 *{4.00}***	(4.15)* *{3.94}***	*{4.20}***	Permanent Instruments
with added mass and masonry infills in all stories (without diagonals) Masonry scheme 2, Planned for 1996	*{6.33}***	*{6.07}***	*{8.52}***	

( )* Assumed experimental value. No y-y translational pull-out was performed

{ }** Numerical simulation results. The measured values of the Young's modulus for the concrete were adjusted so that their maximum becomes equal to 2100000t/m2; moreover, the cross-sections were assumed un-cracked but taking into account the reinforcement.

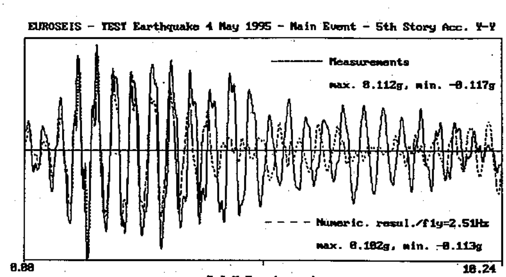
Measured and Predicted Response During the Earthquake of 4th May, 1995 : Two earthquakes occurred on the 4th April, and 4th May 1995, with an epicentral distance from the Volvi test site approximately 40Km. Despite the fact that these earthquakes generated relatively low-intensity ground motion at the test-site, The 5-story structure was excited from those two earthquake 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 recorded acceleration response in the x-x and y-y directions at the 5th floor are depicted in figures 4a and 4b, respectively.

<u>Numerical Simulation of the Observed Earthquake Response</u> : The numerical simulation described before was employed in order to predict the earthquake response of this structure. From the frequency domain study of the response, measured during this earthquake, an adjustment was introduced in the stiffness properties in such a way as to result in the following fundamental frequency values. Translational x-x direction 2.496Hz (instead of 2.603Hz of Table 5); translational y-y direction 2.512Hz (instead of 2.62Hz) and torsional 2.50Hz (instead of 2.53Hz). From a parametric numerical study conducted for this purpose, better agreement between numerical and measured response could be obtained by employing this new set of stiffness. The time history of the earthquake record that was used as input in this numerical simulation that was the one recorded by the 16bit accuracy accelerograph operated by Institute of Engineering Seismology and Earthqauke Engineering at the utility room at a distance of 12m from the foundation block of the 5-story structure. Initially, in this simulation a value was assigned to damping ratio equal to 2.2%, based on the study of the results obtained during the free vibration tests. However, from the numerical parametric study a better agreement could be obtained between the observed maximum earthquake response at the top slab of the 5-story structure and the corresponding results obtained from this numerical simulation when this damping ratio value was taken equal to 3.5%. This increase in the damping ratio value is well justified from the fact that the structure during the free vibration tests was forced at lower levels than during the earthquake sequence of 5th May, 1995. During this sequence the peak horizontal acceleration at the top slab exceeded the level of 0.1g (see figures 4a and 4b). The predicted 5th story acceleration response is compared with the one observed in these same figures, whereby, as can be seen, good agreement is achieved.



f x = 2.49Hz
- - - - Numerical Results, max 0.093g, min -0.094g
____ Measurements, max 0.090g, min -0.094g

Figure 4a. 5-story Structure at Volvi,
5th story x-x acceleration response



f y = 2.51Hz
- - - - Numerical Results, max 0.102g, min -0.113g
____ Measurements, max 0.112g, min -0.117g

Figure 4b. 5-story Structure at Volvi,
5th story y-y acceleration response

7. DISCUSSION OF THE RESULTS - CONCLUSIONS

a. The construction and the instrumentation of the 5-story structure at Volvi, despite the difficulties encountered, was successful. Through the low-amplitude dynamic tests considerable number of data was obtained that was used in deriving the fundamental dynamic characteristics of the structure, assumed to respond in the linear range. This was done for all the structural configurations, presented in this paper, with or without the steel diagonal cables, thus following the variation in the stiffness of the structure introduced by the added components. Moreover, the response of the 5-story model, with added weight and diagonals but no masonry infills, was successfully recorded by the permanent instrumentation during an earthquake sequence of low to moderate intensity (4^{th} May, 1995).

b. The numerical simulation of the dynamic characteristics of the 5-story Volvi building without masonry infills is very successful. This fact must be attributed to the very effective control of micro-cracking 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. However, it must be borne in mind that the measurements used and the assumptions employed in the simulations are based on linear-elastic response.

c. The modal analysis of the asymmetric structural configuration with the diagonal cables, which was performed following the approach outlined in section 5, was successful in identifying almost all 15 modes. Moreover, the numerical simulation of the dynamic characteristics of this asymmetric structural configuration, with the use of the stiffness properties of the symmetric cases, was also very successful.

d. The successful subsequent numerical simulation of the 4^{th} 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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