NUMERICAL SIMULATION OF THE BEHAVIOUR OF A TWO LEVEL STRUCTURE SUBJECTED TO TRANSIENT SEISMIC ARBITRARY LOADS

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Abstract. Numerical 3D finite element method (FEM) of a 2 level frame structure subjected, at the base, to transient arbitray seismic loads is presented in this study. The structural model consists of 2 level structure with constant height of 350 mm and the width of each plate 300 mm. The plates are made out of plexiglass plates to create a rigid plan system and the columns are circular hollow brass rods of diameter 4.00 mm and thickness of 0.40 mm. Free vibration 3D simulation tests are performed in order to investigate the damping properties behavior of the structure. The FEM simulations determines the fields of displacement and normal modes. In order to determine the damping of the structure, especialy the damping of the brass rods, comparisons between experimental and computed free vibration of the structure were done.

Key words: seismic loads, earthquake engineering.

1. INTRODUCTION

As more and more civil engineering metallic structure are build, the numerical 3D FEM is a valuable tool for evaluating more precise the behaviour of such structures under transient seismic arbitrary loads.

In this paper a laboratory 2 level structure with 4 hallow brass rods and level 0, 1 and 2 with plexiglass plates. The levels are of constant height of 350 mm and the width of each plate 300 mm. The structure is shacked at the base with transient displacements in the Ox direction, and displacement in the center of all levels are recorded and compared.

To obtain a correct behaviour of such structures under transient seismic loads, correct elastic material parameters must be put in the numerical simulation, as well as correct damping parameters of the supporting rods.

To obtain the precise elastic material parameters the structure was experimentally subjected to forced vibration by a frequency sweeping in a range of 1Hz–10Hz, and the resonance frequencies were compared with the ones 3D FEM computed. The elastic properties were iteratively adjusted to achieve a good match

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between the experimental and computed results. They were computed in [1] but recomputed 3D FEM with a better mesh in this paper.

To obtain the precise damping parameters of the supporting brass rods the base of the structure was kept fixed and free vibration of the structure were experimentally measured and compared with numerical results. The damping properties were iteratively adjusted to achieve a reasonable match between the experimental and computed results.

In the 3D FEM transient arbitrary loads a time domain analysis was done. Two type of transient displacements applied to the base of the structure were considered:

1) a sinusoidal modulated gaussian signal, with a frequency before the resonance frequency, followed by a frequency in the close vicinity of the resonance frequency, and finally a frequency after the resonance frequency (1 Hz, 2.23 Hz, 3.2 Hz).

2) an arbitrary seismic load, one of a real seismic signal: The Vrancea earthquake of March 4, 1977, with displacements integrated from accelerograms that can be inputted to the 3D FEM computations.

2. NUMERICAL METHOD

The numerical model used in this paper is the 3D FEM, implemented in the software Comsol 5.6, under the Solid Mechanics module [3]:

$$\rho\frac{\partial^2 u}{\partial t^2}=\nabla\cdot S,\ S=S_q+C:\varepsilon,\ \varepsilon=\frac{1}{2}\left[\left(\nabla u\right)^T+\nabla u\right],\ C=C\left(E,\nu\right),$$

$$S_q=\eta_b\dot{\varepsilon}_{vol}+\eta_v\dot{\varepsilon}_{dev},\ \varepsilon_{vol}=\varepsilon_{kk},\ \varepsilon_{dev}=\sqrt{\frac{2}{3}dev\left(\varepsilon\right)_{ij}dev\left(\varepsilon\right)_{ji}}, \tag{1}$$

$$dev(\varepsilon)_{ij}=\varepsilon_{ij}-\frac{\varepsilon_{kk}}{3}\delta_{ij},$$

where u is the displacements vector, S is the stress tensor, ε is the strain tensor, C is the elasticity tensor, ":" stands for the double-dot tensor product, E the Young modulus, ν is the Poisson ratio, ρ the mass density.

S_q denote the viscous damping with two coefficients η_b and η_v representing the bulk and shear viscosity, and ε_{vol} and ε_{dev} are the volumetric and deviatoric parts of the elastic strain tensor.

As the materials are isotropic, the unknowns that are to be determined are the elastic properties of the materials: the Young modulus E, the Poisson ratio ν, the mass density ρ, and the viscous damping properties by the two coefficients η_b and η_v, the bulk and shear viscosity.

3. RESULTS

In order to have good realistic behaviour of the structure subjected to arbitrary seismic loads, the elastic properties of the structure's material parameters as well as the correct viscous damping parameters of the structure, especially the damping of the brass rods, experimental measurements were done.

The structure consists (figs.1,2,3) of 2 level frame made of plexiglass supported by hollow brass rods and an additional plexiglass level (level 0) that attach the structure to the vibration system, providing also the stability for the supporting rods. The plexiglass plates were positioned at level of 350 mm high one from another. Additional steel plates were attached with screws to the plexiglass plates to give a more realistic approach of weight of real existing structures. The steel plates have been manufactured to weight 0.5 kg each, with geometric dimensions of $160\text{ mm}\times 80\text{ mm}\times 6.2\text{ mm}$. The geometric dimensions of the plexiglass plates are $300\text{ mm}\times 300\text{ mm}\times 4\text{ mm}$, with different holes for the supporting rods and additional steel plates. The supporting rods are positioned to 40 mm of the plexiglass plates edges, and not less, to ensure that during the vibrations the plexiglass will not break. The geometric dimensions of the supporting hollow brass rods are the exterior diameter of 4 mm and the inner diameter of 3.2 mm. The total length of the rods are 1 m, the purchased length, and kept so, for subsequent experiments, that will support additional plates of plexiglass. To ensure that the supporting rods will not collapse during the vibrations, as well as a precision mounting of the plexiglass plates, stiffener assemblies, fig. 4, made up of washers and nuts were used to attach the plexiglass plates to supporting rods.

The displacements measuring system, fig. 5, consists of displacements and acceleration sensors, amplifiers with integration facilities, fig. 9, and an oscilloscope, fig. 10, connected to a computer for data acquisition. One displacement sensor of type Celesco SP2-4 was used as a permanent reference of the displacements measured on the base of the structure and connected to the vibration system. Other 4 acceleration sensors, named usually accelerometers, were used to measure the displacements in various points of the vibrating structure by integration of the accelerations with amplifiers having integration facilities. Of the 4 accelerometers, one is of type Bruel&Kaer Charge Accelerometer 4381, and named by us Type 2 along this paper, and the others 3 are of type HMF KB12, and named by us Type 1. The 3 amplifiers with integration facilities are Bruel&Kaer Charge Amplifier type 2635. The oscilloscope used was an Agilent DSO5014A, with 4 channels, 300 MHz bandwidth on 16 bits. The first channel was used to connect to the displacement sensor and used as a reference signal for the others. For the remaining 3 channels, only 3 of the 4 accelerometers were used at one time.

Fig. 1 –The laboratory structure analyzed.

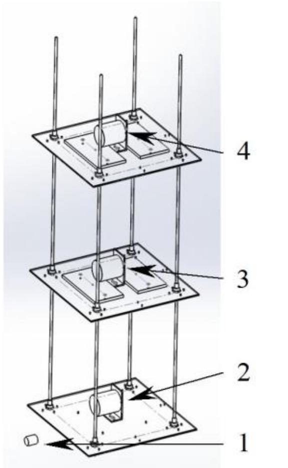

Fig. 2 – Configuration of sensor locations connected to 1,2,3,4 channels of the oscilloscope.

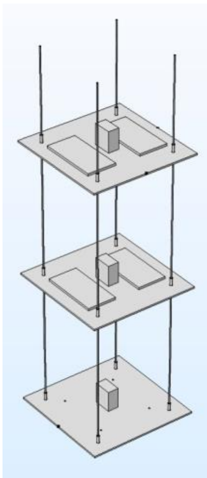

Fig. 3 – Simplified structure taken in 3D FEM.

Fig. 4 – Stiffener with washers and nuts.

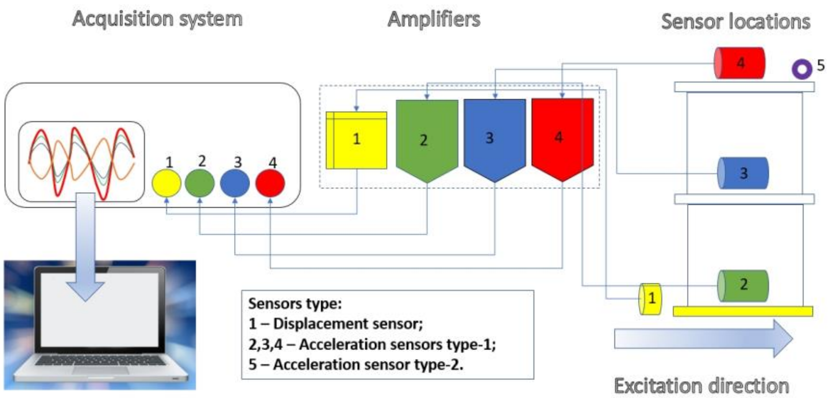

Fig. 5 – Schematic diagram of the experimental data acquisition system with sensors.

Fig. 6 – Celesco SP2-4 displacements sensor.

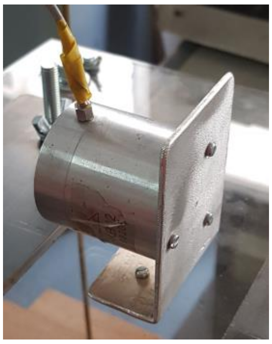

Fig. 7 – Charge Accelerometer HMF KB12.

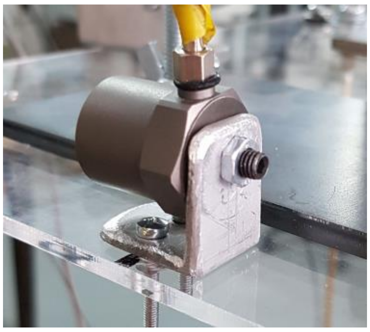

Fig. 8 – Bruel&Kaer Charge Accelerometer 4381.

Fig. 9 – Bruel&Kaer Charge Amplifiers Type 2635.

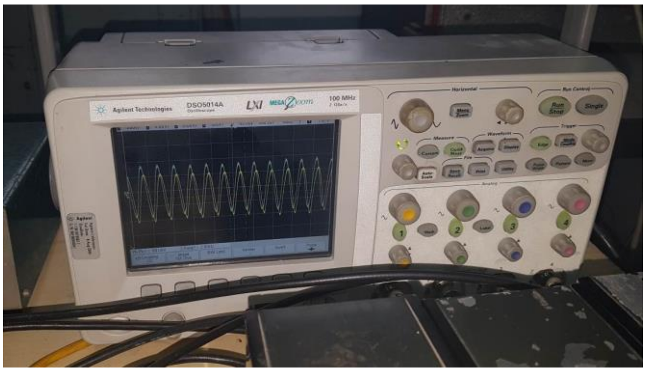

Fig. 10 – Acquisition system oscilloscope Agilent 5014A.

The elastic properties of the structure's material parameters were determined in [1], by comparisons between experimental and computed results in the frequency domain. In [1], as a first step an experimental calibration of the complete measurement system, was done. It consists in calibration of the amplifier settings according to each sensor used, by mounting all the 3 accelerometers on the base of the vibration table and comparing with the signal given by the reference displacement sensor, at various vibration frequencies of the structure. In [1] the elastic properties of the structure's materials were determined to be:

- for brass: $E = 111\ \text{GPa}$, $\nu = 0.335$, $\rho = 8940\ \text{kg/m}^3$;
- for plexiglass: $E = 3.0\ \text{GPa}$, $\nu = 0.4$, $\rho = 1190\ \text{kg/m}^3$;
- for steel: $E = 211.9\ \text{GPa}$, $\nu = 0.228$, $\rho = 7860\ \text{kg/m}^3$;
- for aluminum: $E = 73.14\ \text{GPa}$, $\nu = 0.331$, $\rho = 2780\ \text{kg/m}^3$.

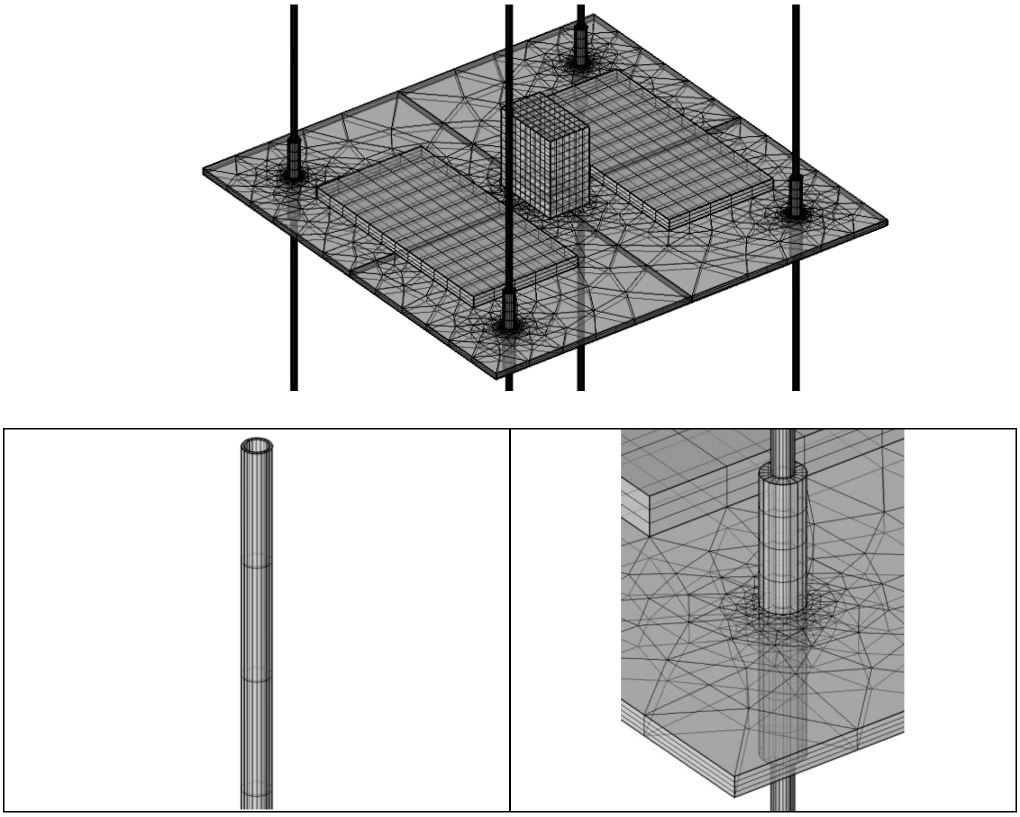

Fig. 11 – Mapped mesh considered in 3D FEM simulations.

As the time domain analysis require much more computational resources than the eigenfrequency or frequency domain analysis a simplified structure, fig. 3, of the real structure, fig. 1, was taken into account. Also a better mesh, fig. 11, with mapped elements was considered. The number of elements was reduced from 475341 domain elements in free "extra fine" triangular mesh to 28576 domain elements, for similar results.

The correct viscous damping parameters of the structure, especially the damping of the brass rods, are determined by comparisons between experimental and computed results in the time domain of free vibrations.

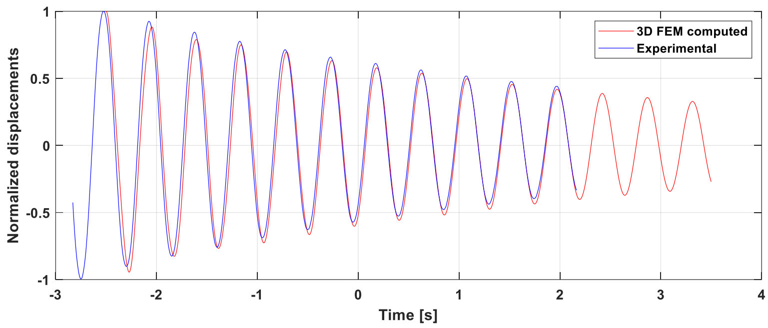

Fig. 12 – Measured signal during the experimental evaluation of the structural damping.

Experimentally the base of the structure was considered fixed and a force was applied to level 2 of the structure, after which it was allowed to vibrate freely, as described in [1]. The experimental determination of the structural damping was performed by a free vibration test and the measurement of the variation of the measured displacements over time at level 1 and level 2 of the structure, revealing so the exponential decrease of the amplitude vibrations as well as the resonance frequency of the structure, fig. 12.

For underdamped vibrations [6], the damping ratio ς is related to logarithmic decrement δ:

$$\zeta = \frac{\delta}{\sqrt{\delta^2 + (2\pi)^2}}, \tag{2}$$

with $\delta = \ln\frac{x_0}{x_1}$ where x_0 and x_1 are amplitudes of any two successive peaks.

This experimental evaluation of the structural damping ratio was determined that the damping ratio is 1.3226%, and the resonance frequency is $2.2263\ \text{Hz}$.

In the 3D FEM the viscous damping coefficients η_b and η_v representing the bulk and shear viscosity were found by iterative simulations, $\eta_b = 0$ and variable η_v, until the simulated logarithmic decay match the experimental one. The coefficient η_b was considered $\eta_b = 0$ as it has little influence, and only η_v was considered variable. The analysis consists of a 2 steps, as in experimental measurements, the first one a stationary analysis with a force applied horizontaly in the Ox direction to the second level of the structure, and a second analysis a time domain one, simulating the free vibrations of the structure.

As in the experimental measurements and in the computed results the logaritmic decrement δ had significant differences from one sinusoid to another, it was averaged over the 10 sinusoids measured and computed. It was found that shear damping coefficient $\eta_v = 32 \cdot 10^6$ Pa $\cdot$ s is the best match for the studied structure.

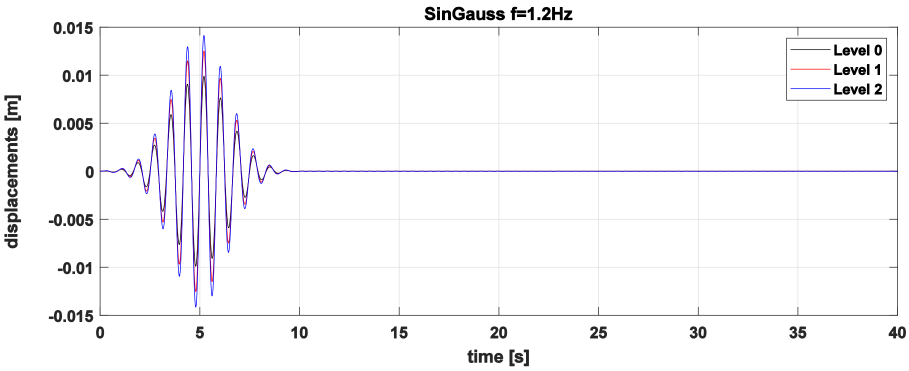

Fig. 13 – The displacements of the structure in the 0x direction subjected to a sinusoidal modulated gaussian signal with the frequency 1.2 Hz applied to the base (level 0) of the structure.

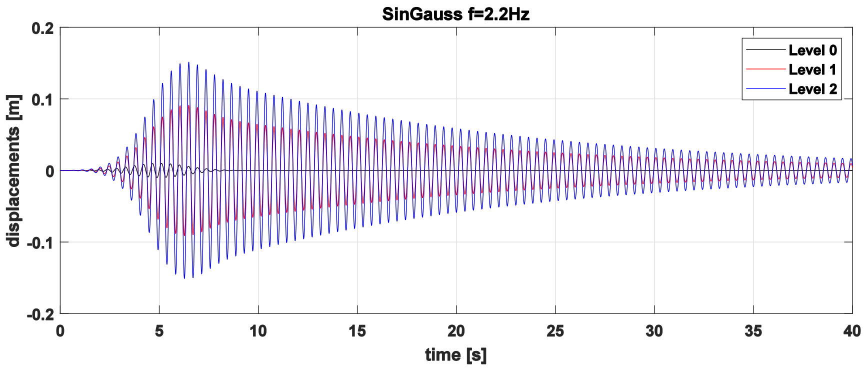

Fig. 14 – The displacements of the structure in the 0x direction subjected to a sinusoidal modulated gaussian signal with the frequency 2.2 Hz applied to the base (level 0) of the structure.

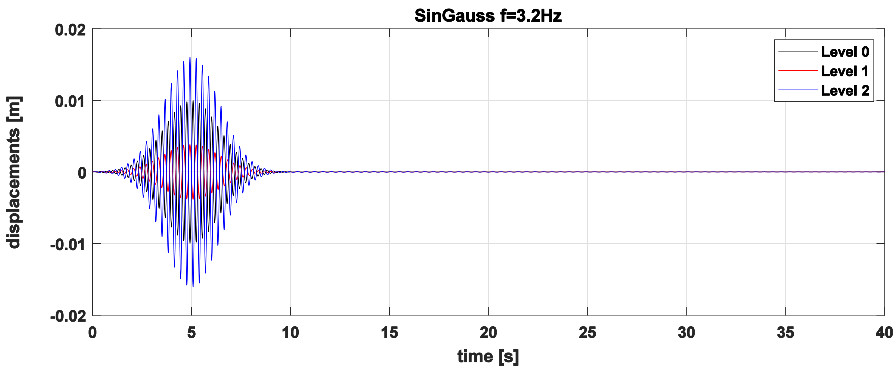

Fig. 15 – The displacements of the structure in the 0x direction subjected to a sinusoidal modulated gaussian signal with the frequency 3.2 Hz applied to the base (level 0) of the structure.

With these elastic and damping material properties correctly determined from eigenfrequency, frequency domain and free vibrations time domain analysis, some time domain analysis with transient arbitrary seismic loads applied to the base of the structure in the Ox direction were applied.

A first type of time domain analysis with arbitrary seismic loads, was a sinusoidal modulated gaussian signal, with a frequency before the resonance frequency, followed by a frequency in the close vicinity of the resonance frequency, and finally a frequency after the resonance frequency (1 Hz, 2.23 Hz, 3.2 Hz).

The sinusoidal modulated gaussian signal was considered to be a 10 s signal delayed with 5 s:

$$f(t) = A \cdot e^{-c\cdot(t-5)^2} \cdot \sin\left(2\pi f\left(t-5\right)\right)$$

where A is the signal amplitude, c a sharpness coefficient and f the frequency of the sinusoidal signal.

In our simulation the amplitude was considered to be $A = 2$ cm (4 cm peak to peak), and $c = 0.25$. The active signal acting on the base of the structure was considered to be 10 s long and null after, and the simulated time of the structure's behaviour 50 s. The behaviour of a two level structure subjected to these transient arbitrary loads at frequencies f=1.2 Hz, 2.2 Hz, 3.2 Hz are plotted in figs.13, 14, 15.

It can be seen that, as expected, the frequency 2.2 Hz of the sinusoidal gaussian modulated signal make the structure to enter in resonance.

A second type of time domain analysis with arbitrary seismic loads was the one of a real seismic signal given by the free version of the software SeismoSignal [5]. Usually the data measured on earthquakes are accelerograms because they are easy to record and very sensitive. The accelerograms can be easily integrated numerically and displacements are computed by finite differences by approximation of integration constants [4]. Also the SeismoSignal software can compute the displacements that can be used in simulations.

We took as example the Vrancea earthquake of march 4 1977. From the recorded accelerograms, fig.16.a), by numerical integration [4], velocities, fig.16.b), and displacements, fig.16.c), can be found. However these displacements are for real structures, not scaled for lab measurements. So we divided the displacements by 10 and introduced them in our simulations. By doing so, the shape of the accelerograms remain the same, but their level decrease 10 time. To have the same level of accelerations, and so the same level of forces applied to the structure, the time was also divided by 10. In fig.17 and fig.18 can be seen the computed behaviour of the studied structure (ground level (level 0), level 1 and level 2), in two computational cases: the laboratory scale and the real scale. It can be seen that the computed displacements of the structure at laboratory scale and real scale are similar, and so the computed results at laboratory scale are a good approximation of the ones at real scale.

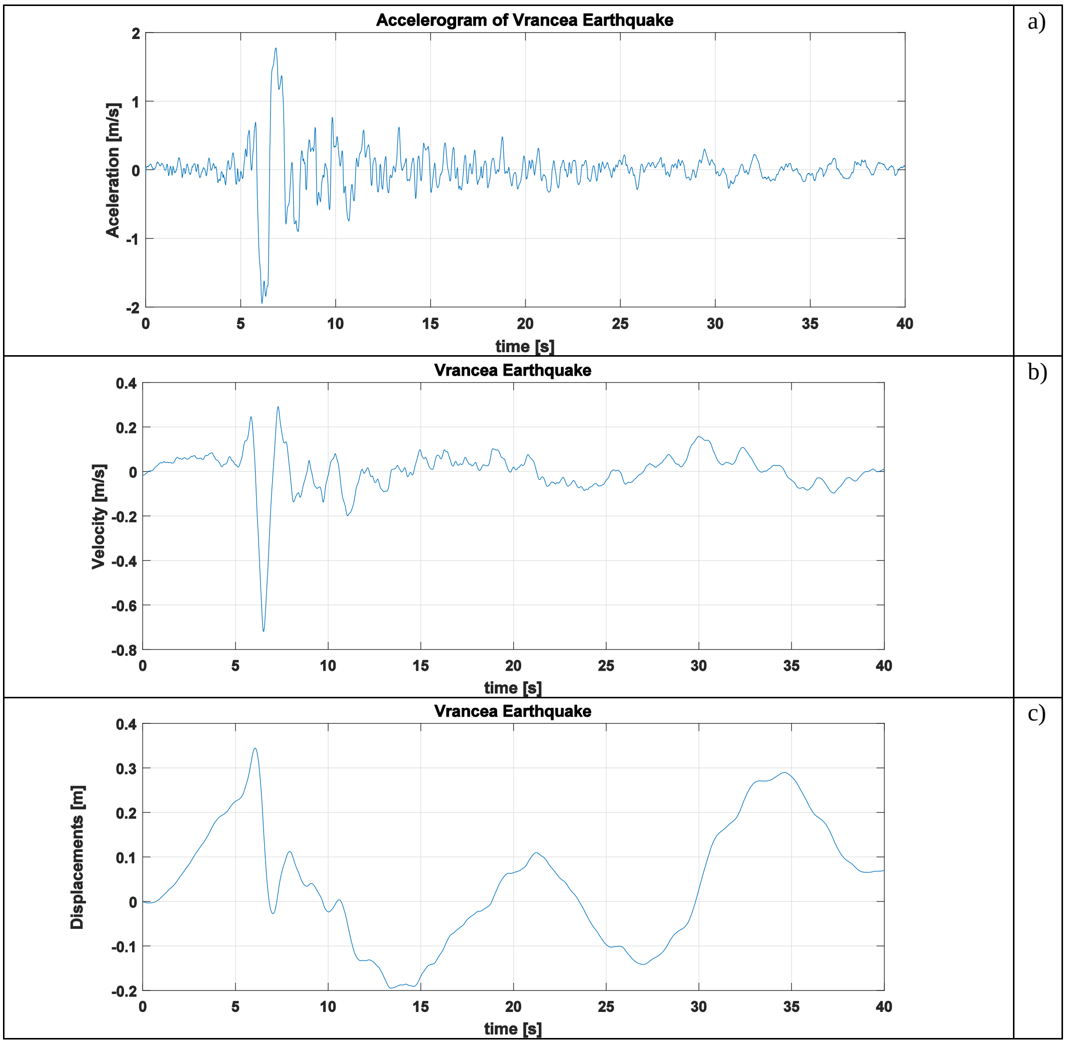

Fig. 16 – The Vrancea earthquake of 1977, with recorded accelerogram (a) integrated velocity (b) and integrated displacements (c) by integration method presented in [4].

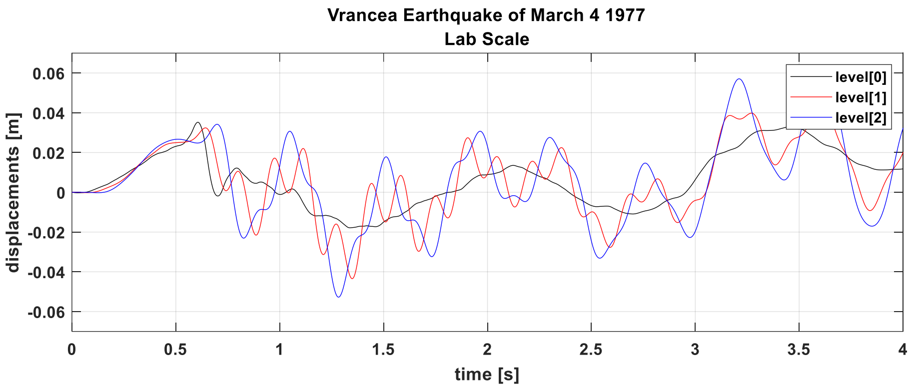

Fig. 17 – Computed response of the structure at laboratory scale of the March 4 1977 Vrancea earthquake.

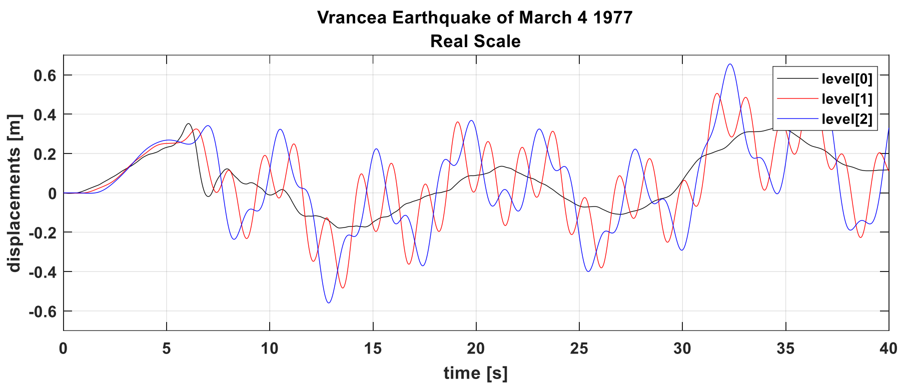

Fig. 18 – Computed response of the structure at real scale of the March 4 1977 Vrancea earthquake.

3. CONCLUSIONS

The behaviour of a laboratory real two level structure subjected to transient seismic arbitrary loads was 3D FEM analyzed.

The elastic and damping material properties taken in transient 3D FEM computations were adjusted to match the experimental forced vibration data as well as experimental free vibration data measured on the structure.

By numerical integration of accelerograms, the displacement can be successfully used to compute real structure behaviour. By reducing the geometrical dimensions of the real structure by 10 and the earthquake time also by 10, the behaviour of a laboratory scaled model approximate the real structure behaviour reasonably well.

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