High-rise building dynamics identification through shaking table measurements on scale model for multi-hazard experiments

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ABSTRACT
In the field of civil engineering, multi-hazard experiments are largely used to study the structure interaction under different extreme environmental conditions. This paper discusses the laboratory experiment needs to investigate the double interaction between earthquake-structure and wind-structure for a high-rise building used as case of study. In particular, the dynamics parameters of an in-scale model was identified though measurements on shaking table. The same model will be used for experiments in wind tunnel. Results have shown that the very small scale of models needed for wind tunnel experiments affects the structural identification in particular for the damping ratio estimation. The uncertainty propagation on the pseudo-acceleration spectrum due to the damping ratio variability was discussed.

1 INTRODUCTION

According to Emporis, partner of the Council on Tall Buildings and Urban Habitat (CTBUH) in Chicago (https://www.emporis.com), there are about ten thousand high-rise buildings with a height greater than 200 m, in the first 50 most populous cities in the world. This number is expected to increase in the future due to both soil saving and an increasing world population. Many of these buildings were built in areas with a high seismic risk and for this reason investigations concerning earthquake-structure interaction cannot be neglected even if these structures generally have high natural periods of vibration. At the same time, they are very sensitive to wind action which is worsened by their very unique shapes and architectural designs (Smith and Coull 1991). As is well known, the wind-structure interaction is investigated though experiments in wind tunnels and computational fluid dynamic simulations. In the case of experimental tests in wind tunnels, aerodynamic and aeroelastic in-scale models are designed in order to investigate the pressure coefficients (i.e. aerodynamic models) and dynamic effects (i.e. aeroelastic models). Similarly, tests on shaking table are carried out on models to investigate the earthquake-structure interaction (Ferrareto et al 2016). Generally, the two experimental models, i.e. the setup used for wind tunnel test and the other used for shaking table experiments, are not the same since the scale sizes that the two experiments need are not the same. Usually, the wind tunnel test chamber sizes are small; the most common are smaller than 3m x 3 m which means that the model scale has to be small in order to avoid flow blockage (Isyumov 1982). In the case of aeroelastic tests, the small geometrical scale affects all experimental processes because it requires a precise mass and stiffness sizing. With this background the simulation of the damping ratio becomes the most difficult phase and it is very likely that the experimental error will affects experiment results (Rizzo and Caracoglia 2018, Rizzo et al 2018, Brito and Caracoglia 2009).

On the contrary, models for the shaking table are generally bigger than wind tunnel models, which simulates masses, stiffness and structural details precisely (Maddaloni et al 2011, 2012 and 2017, Caterino et al 2015). In order to estimate the multi-hazard effects under the contemporary action given by wind and earthquake, experiments should be carried out on the same in-scale model. In particular, seismic accelerations should be given during wind tunnel tests. The case of study herein discusses structural design and model scaling for a high-rise building, including in-scale model dynamic identification and its response under seismic impulse on shaking table experiments.

2 CASE OF STUDY AND EXPERIMENTAL SETUP

2.1 High-rise building structural sizing.

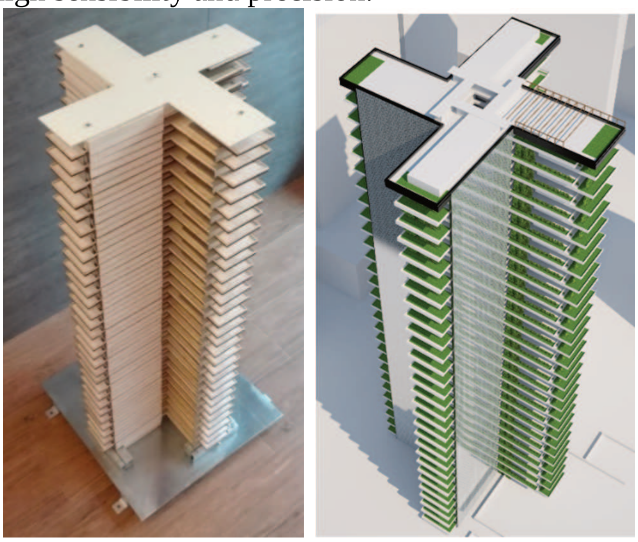
The building is 300 m tall, and its floor plan is inscribed in a 138 m side square. The building has 60 floors and each floor has a height (*h*) equal to 5.00 m; this is the combination of a free floor-to-ceiling height of 3.50 m and a floor thickness, including the equipment, of 1.50 m. The building consists of four, asymmetric wings. In the centre, there is a large (14 m by 14 m) open square space, linking the four wings together. The balconies are cantilevered slabs which have two alternating geometries, as shown in Figures 1 a) and b). The geometry is more prominent along the transversal direction of the wing, the other is more prominent along the longitudinal direction. The very long cantilevered slabs (i.e. between 8 m and 10 m) are expected to affect the aerodynamic and aeroelastic response. The large cantilevered slabs do not allow diagrid systems, and the large open space located in the middle of the building does not allow for a central concrete rigid core. For this reason, a more traditional framed system with rigid shear slabs and shear walls had to be adopted, including shear steel trusses and shear reinforced concrete walls. Interior floor space alternates different intended uses and for this reason, there is variable distribution of live loads along the building height.

Structural analyses on a FEM model was carried out to design structural elements and to estimate the building natural frequencies according to CEN 1991 and CEN 2005. In this phase, seismic action was modelled according to the response spectrum analyses (CEN 2005) and the wind loads were applied as static equivalent loads according to CNR DT 207.

The structural element sizing will be calibrated after experimental tests in wind tunnel and on shaking table. The process is iterative and aims to optimize the structure so as to avoid undesirable aeroelastic effects and vibrations.

2.2 In-scale model design.

One of the most difficult aspects to investigate multi-hazard effects on a High-rise building, is the geometrical scale of the experimental model. Generally, it is affected by the wind tunnel test chamber sizes. In fact, in order to avoid flow blockage, the model sizes have to be quite small which represents a problem because a small geometrical scale affects model precision and its correct distribution of masses and stiffness. In addition, tests on the shaking table have to be carried out through an analog control system with high sensibility and precision.



(a) in-scale model view. (b) rendering model view
Figure 1. High-rise building

In this case, the model was designed for being tested in the wind tunnel situated in the Ancona Polytechnic University, Italy. The cross-sectional dimensions of the test chamber are 1.80 m by 1.80 m. A geometric scale λ_L = 1:400 was chosen, which is in the range of 1:500 to 1:300 suggested by Isyumov 1982. In this case the in-scale model is in a prism with a square plan sized 300 mm x 300 mm and a height equal to 750 mm. The expected blockage is between 8% and 11%, which are slightly high values and a correction will be necessary.

In order to reproduce an in-scale model of the prototype, a FE numerical in-scale model was used to check the target dynamic properties of the physical model. The slabs were modelled using 4 mm thick plate elements with a density equal to 538 kg/m^3, corresponding to the value measured for poplar plywood. Horizontal diaphragms were added to reproduce in-plane stiffness, while the four walls were reproduced through 4.75 x 750 x 2 mm aluminium shell elements (E=68.9GPa, γ=2700 kg/m^3). The model was given base pin constrain to reproduce the physical model. All materials are chosen in order to respect the scaling criteria (Isyumov 1882) given by Eqs.(1-3). Modal analyses were carried out to evaluate natural frequency vibration modes and participating masses of the in-scale model structure.

The scaling criteria given by Eqs. (1-3) define the translational (λ_m) and rotational (λ_I) mass ratios between model and prototype, respectively. The subscript m means model, whereas p means prototype. ρ is the bulk modulus that is generally rather similar between model and prototype (i.e. $\lambda_\rho \approx 1$)(Isyumov 1982)

$$\lambda_m = \frac{m_m}{m_p} = \frac{{H_m}^3 \rho_m}{{H_p}^3 \rho_p} = {\lambda_L}^3 \lambda_\rho \qquad (1)$$

$$\lambda_I = \frac{I_m}{I_p} = \frac{{H_m}^5 \rho_m}{{H_p}^5 \rho_p} = {\lambda_L}^5 \lambda_\rho \qquad (2)$$

Velocity scale $\lambda_V$and acceleration scale $\lambda_a$are obtained as:

$$\lambda_V = \frac{V_m}{V_p} = \lambda_L \lambda_\eta \;; \lambda_a = \frac{\lambda_V}{\lambda_L} \qquad (3)$$

where the $\lambda_\eta = \frac{1}{\lambda_T}$ is the frequency scale based on mode 1, (Isyumov 1982). In this case, the first (i.e. horizontal along X) and the second (i.e. horizontal along Y) mode frequencies are close enough to be considered equal.

The damping ratio of the model has to be equal to the prototype value. With Fixed λ_L and consequentlyλ_m, the goal is to predict the desired λ_η in order to have an optimal λ_V. In fact, the full-scale wind speed$V_p$is fixed as a function of the geographical parameters, whereas the in-scale wind speed $V_m$should avoid being damaged during experiments.

In the case of this prototype, the estimated λ_m and $\lambda_{I,m}$ ratios are equal to 1.56·10^{-8} and 9.76·10^{-4} respectively. The $\lambda_\eta$frequency scale is equal to 59.4. Consequently, the $\lambda_V$speed scale is equal to 0.15 and the λ_a acceleration scale is equal to 8.83.

2.3 Shaking table setup.

Experiments on the shaking table were carried out at the laboratory of the Department of Structures for Engineering and Architecture of Federico II University, Naples (Italy). The aim is to identify the modal shapes, natural frequencies and damping ratio of the model.

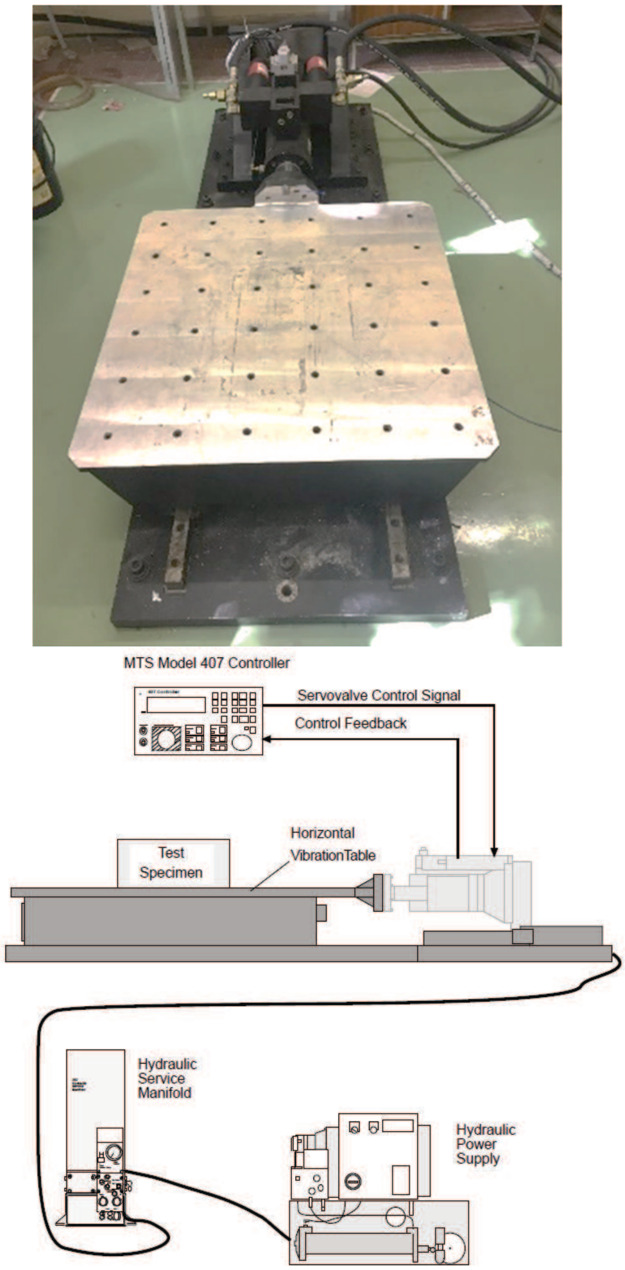


Figure 2. The shaking table and the analog control system.

The shaking table system is composed by: (I) an analog control system which conditions, monitors, and generates program commands and feedback signals for control of the test system;
(II) a linear actuator which allow to shake the platform in a horizontal direction; (III) a hydraulic distribution system which pumps hydraulic fluid to the actuator servo valve from the hydraulic power supply.

The hydraulic actuator is a linear force-generating actuator that operates under servo valve control. Actuator features include:

(I) hydrostatic pressure-centering bearings that react to the high side loads induced by off-center loading on the vibration table;

(II) an internally-mounted linear variable differential transformer (LVDT) to measure actuator displacement;

(III) built-in hydraulic cushions that reduce the speed of the actuator piston when operating outside of the actuator dynamic stroke range or in the event of an open-loop situation. The actuator for the horizontal vibration test system is force rated for 2.2 kip (10 kN) with a dynamic stroke (linear displacement) of 5.9 inches (150 mm).

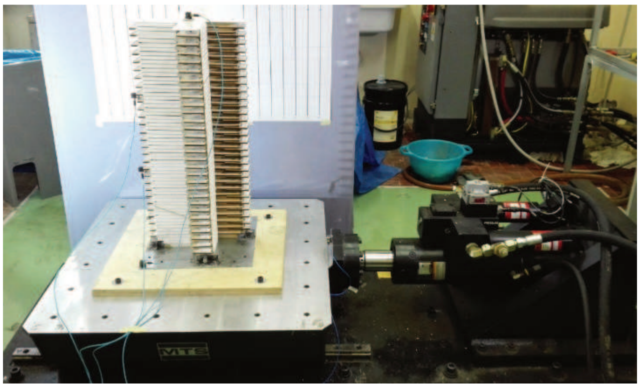


Figure 3. Experimental setup.

The servo valve used with the system is designed for a maximum operating pressure of 3000 psi (21 MPa) and has a 10 gpm (38 lpm) flow rating. The shaking table is constructed to provide high stiffness with minimum possible weight.

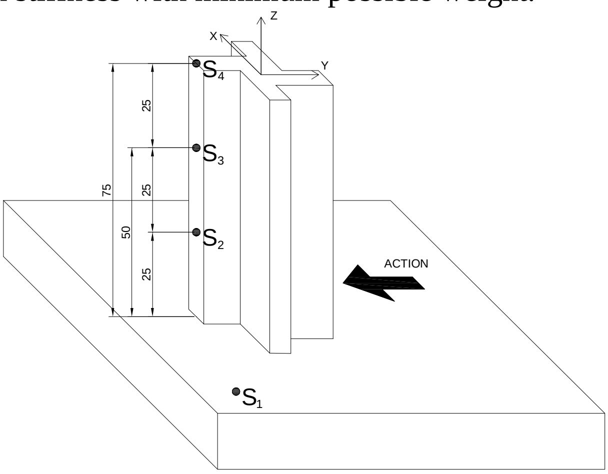


Figure 4. Overview of the accelerometers positions.

Figures 2a and b show a view of the shaking table and a schematic outline of the analog control system. Figure 3 shows a view of the experimental setup. Models were fixed on the shaking table with a 27 mm thick wood plane and anchored through nuts and bolts. Their tightening torque was monitored before and after each measurement.

Figure 4 shows a view of the accelerometer positions. Four accelerometers were placed: on the base of the shake table (S_1), at 1/3 of H (S_2), at 2/3 of H (S_3) and on the top of the building (S_4). The accelerometers used had a weight of 9gm and a size of 14x20x14 mm. The sensitivity was 51 mV/(m/s^2) and the frequency range was 0.5-3000 Hz with a resonant frequency greater than 14kHz.

Different measurements were acquired for different purposes, including estimating the natural frequencies of the model. and estimating the damping ratio. Other measurements under seismic impulse aimed to estimate the acceleration amplification ratio from top to base.

3 DYNAMIC IDENTIFICATION

The dynamic identification of the in-scale model was carried out through two different sets of experiments.

The first set of experiments was focused on estimating the natural frequencies. In particular in this case, only the first natural frequency was investigated because numerical analyses results have shown that the first two natural frequencies are close enough to be considered the same and they have the most participating masses (i.e. 80%). In-scale model natural frequencies were estimated using random acceleration and sine sweep signal inputs with range frequency 0.1 Hz-5.5 Hz. Experiments were repeated 100 times to investigate the experimental error propagation. Figure 5 shows the transfer function from base to top.

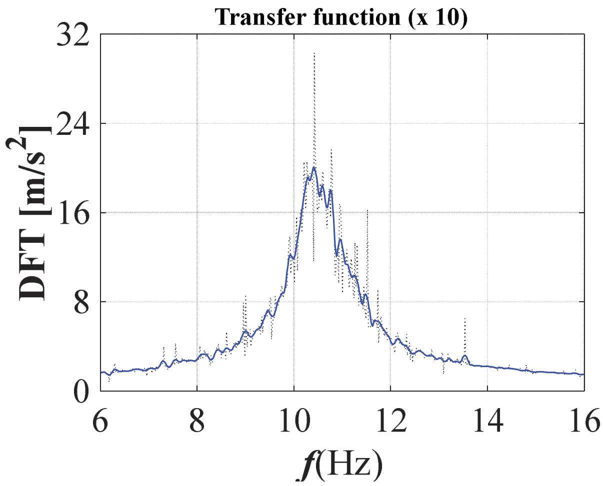
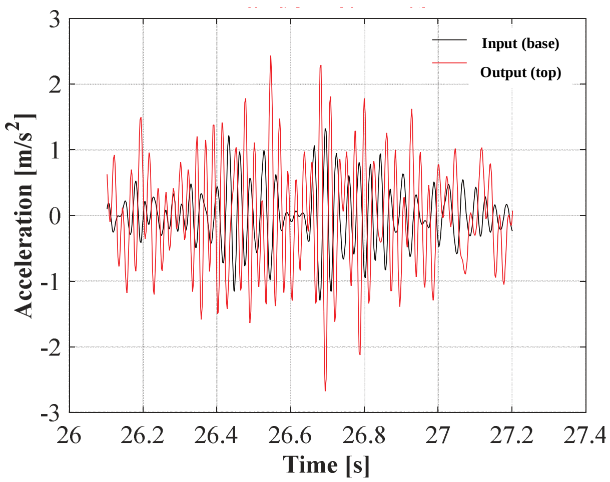


Figure 5. Transfer function.

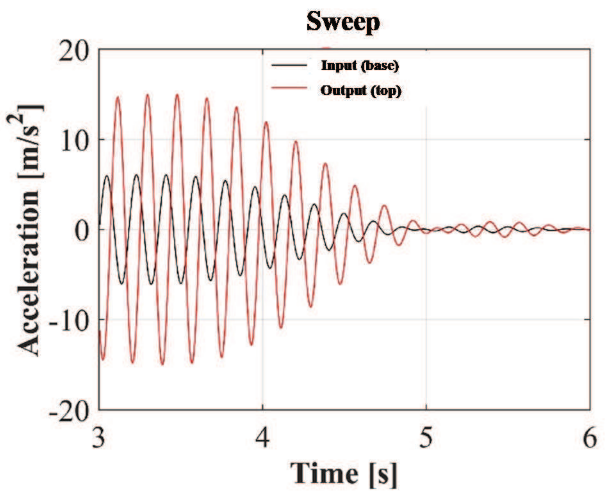
Figure 6a shows a 1.0 s sample time history of the acceleration recorded at the top and at the base of the building by a random shaking table input.

The second sets of measurements aimed to estimate the in-scale model structural damping ratio ξ. This was estimated calculating the logarithmic decrement of the free decay of 20 cycles of sine sweep signal. Also in this case the experiment was repeated 100 times. Figure 6b shows the tail of the recorded base and top acceleration.

The first natural frequency (i.e. mean value of the 100 measurements) is about 10.8 Hz (0.1 s), (with a coefficient of variation equal to 5.6%) that means a value of 0.181 Hz (5.52 s) for the real scale building. The damping ratio (i.e. mean value of the 100 measurements) is about 2.4% (with a standard deviation equal to 0.8% and a CoV equal to 33.3%). The damping ratio uncertainty is closely non-negligible which confirms the difficulty in calibrating this dynamic value using a very small geometrical scale. The effect of this variability is discussed in Section 5.



(a)



(b)

Figure 6. Dynamic identification: Random series (a), free decay sine sweep (b).

However, in absence of specific analyses that confirm the effect of the curtain walls, and in order to investigate the wind-structure interaction in wind tunnel, these values should be reduced. This is an iterative phase which means that shaking table measurements should be repeated too.

4 SEISMIC INPUT ACCELERATION

The accelerations recorded during the L'Aquila 2009 earthquake (PGA equal to 0.66 g) was used as input. The seismic input was scaled according the scaling ratios given in Section 2.2 (i.e. λa = 8.83), assuming a peak acceleration of 0.078g (i.e. 0.66 g/8.83).

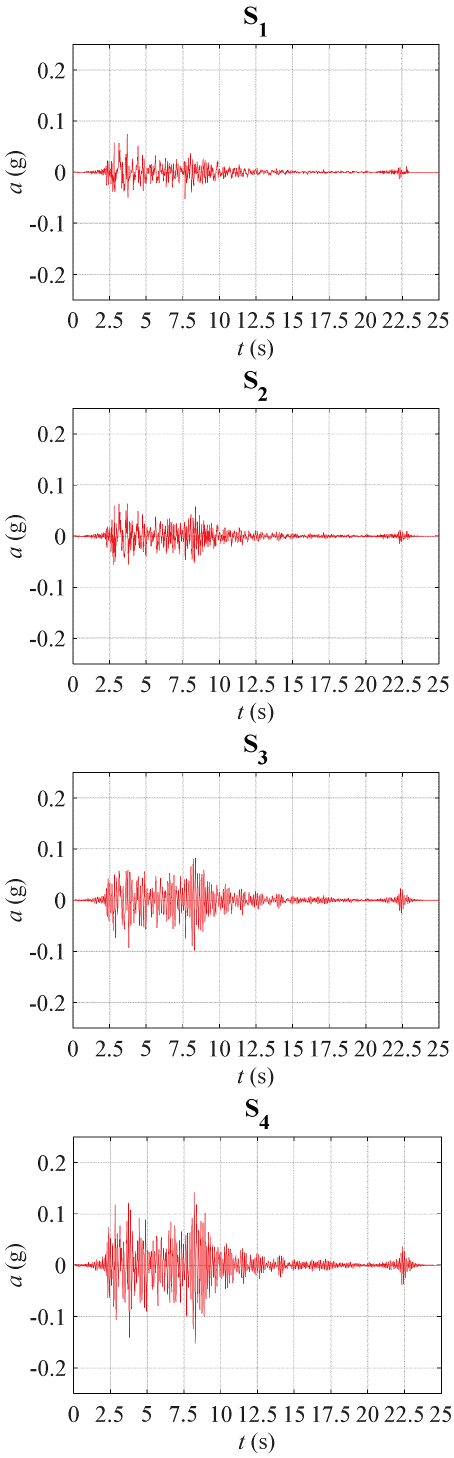


Figure 7. In-scale model response under seismic impulse.

Figure 7 shows the accelerometer signal registrations during experiments. Accelerometers are labelled S_1, S_2, S_3 and S_4 from the base to the top.

The acceleration amplification ratio, A_a, (Mazzolani and Herrera 2012) between the ground peak acceleration (i.e. the maximum shaking table acceleration) measured by S_1 sensor and the peak accelerations measured by S_2 sensor (placed at 1/3 of H building, i.e. at 25 cm from the shaking table base) is between 0.85 and 1.07. The A_a ratio ranges from 1.10 to 1.87 for ground peak acceleration measured by S_1 sensor and S_3 sensor (placed at 2/3 of H building, i.e. at 50 cm from the shaking table base) and, finally, it ranges from 1.90 to 2.91 for peak acceleration measured by S_1 sensor and the S_4 sensor (placed at the top of the building, i.e. at 75 cm from the shaking table plane).

The acceleration amplification ratio (A_a) trend is illustrated in Figure 8. A value of about 3 is obtained for acceleration amplification ratio between the top and base building acceleration. The A_a trend can be fitted by non-linear curves and in particular by second order polynomial curves.

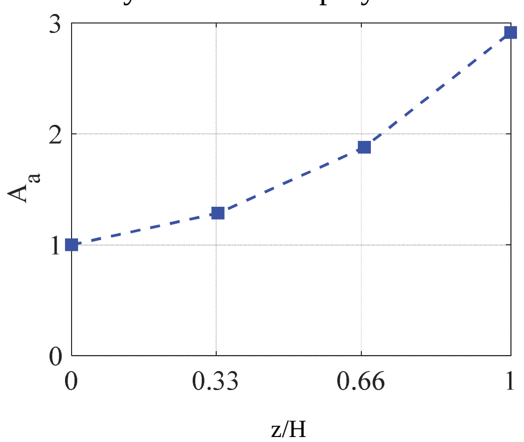


Figure 8. Acceleration amplification ratio.

5 DYNAMIC RESPONSE AT PROTOTYPE SCALE

The floor acceleration signals illustrated in Figure 7, at prototype scale (i.e. $\lambda a = 8.83$) give peaks in the range from 0.46 m/s^2 and 1.34 m/s^2 (0.047g and 0.137g), from the base to top. Velocities and displacements were estimated by single and double integration respectively and subsequently scaled at prototype (real) scale (i.e. the time scale, λt, is about 0.17). Figure 9 shows horizontal displacements at 1/3 H, 2/3 H and H, respectively. A maximum displacement of 0.15 m was recorded at the top of the building. Figure 10 shows the elastic response pseudo acceleration spectrum computed by S_1, S_2, S_3 and S_4 signals. The peak of the pseudo acceleration spectrum is equal to 10.5 m/s^2 at 0.15s.

The structural first natural period at prototype scale is equal to 5.52 s and it corresponds to a neglected values of pseudo acceleration spectrum (i.e. 0.007 m/s^2).

Consequently, through double integration the pseudo velocity and displacements spectrum were estimated with a damping ratio equal to 2.4% and period range between 0 and 10 s. Results are displayed in Figures 11 and 12.

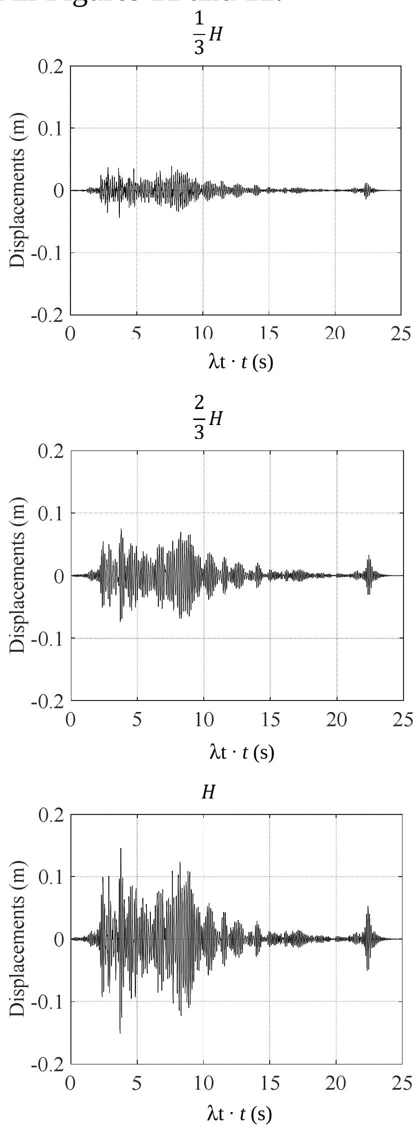


Figure 9. Prototype scale floor displacements (i.e. x-axis is scaled

As it was expected, the peak of the pseudo acceleration spectrum is for 0.15 s and it is very distant from the structural natural period (i.e. 5.5 s). The pseudo acceleration spectrum peaks varies from about 28 m/s^2 to 102 m/s^2 from base to top.

Figures 10, 11 and 12 show a zoom in the range between 5 and 6 s. Figures show a ratio equal to about 1.16 between base to top.

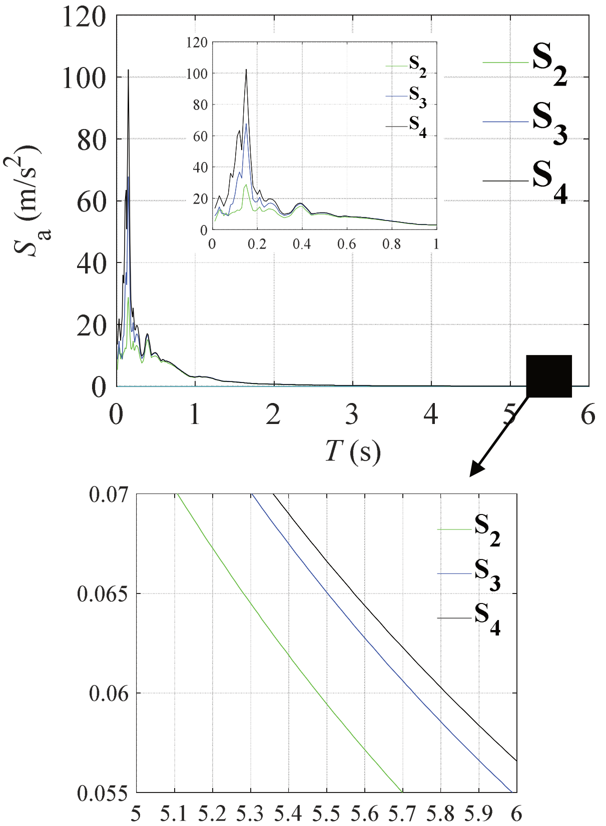


Figure 10. Elastic response pseudo acceleration spectrum

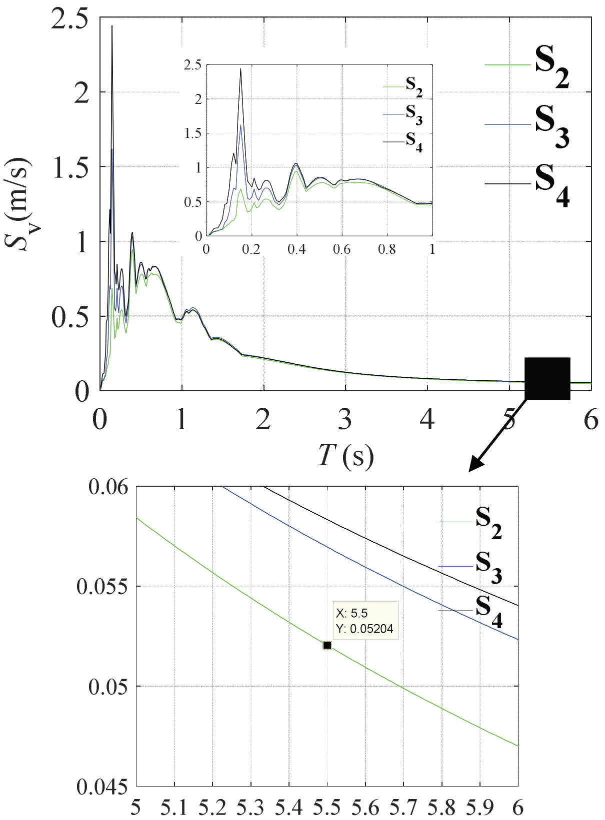


Figure 11. Elastic response pseudo velocity spectrum

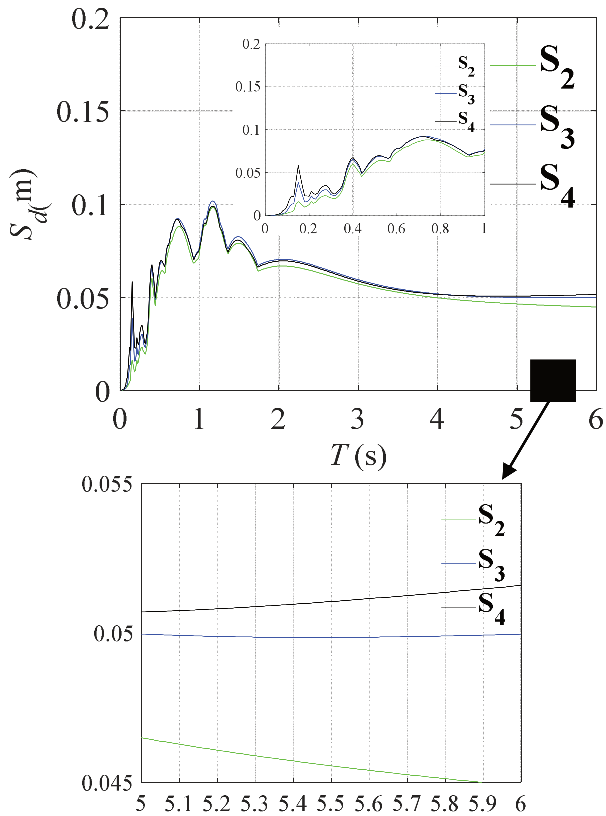


Figure 12. Elastic response displacements

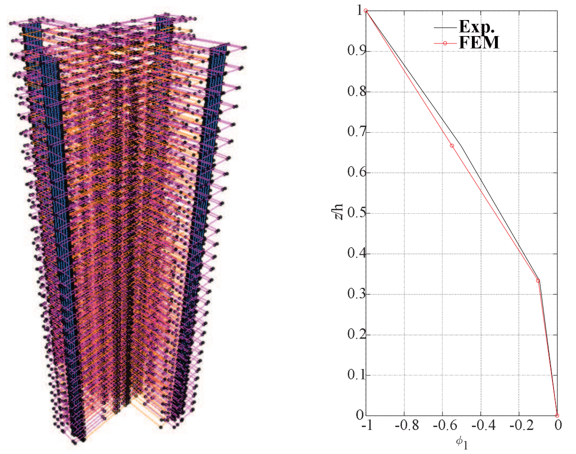
It was noted that the peaks accelerations base to top are significantly bigger than measurements given by Reinoso and Miranda 2005. However, a believable explanation is that buildings assumed as case of study by Reinoso and Miranda 2005 are generally very distant from the earthquake epicentre.

In fact, buildings in San Francisco Bay Area were distant about 90 km from epicentre of the 1980 Loma Prieta earthquake, and about 30 km from 1994 Northridge earthquake epicentre. In the case of Reinoso and Miranda 2005 the peak ground accelerations ranges from 0.1 to 0.2g. As it was discussed in Section 4, 2009 L'Aquila earthquake peak ground acceleration is 0.66g at prototype scale. It justifies the significant differences between spectra illustrated in Figures 10,11 and 12.

Results allow to exclude significant vulnerability under seismic impulses. However, this study is focused on the wind-seismic interaction and it is reasonable to think that accelerations and displacements will be affected by the multi-hazard combined action between wind ad earthquake.

In order to calibrate the wind tunnel experimental setup, based on shaking table experiments results, a numerical in-scale FE model was computed. The numerical model was calibrated based on the first natural frequency and model shape. In addition, the in-scale model stiffness and masses were correctly reproduced.

Figure 13 shows the estimated and the numerically reproduced first mode of vibration. The modal shape is shown as a function of the *z*/h ratio and the mode shapes of vibration, ϕ_i.



(a) FEM in-scale model (b) First Natural mode.

Figure 13. In-scale numerical model.

Future improvements aim to estimate wind-seismic fragility curves that are useful to predict the damage probability for the building similar to those studied (Alwaeli et al 2004).

Finally, in order to have a measure of the uncertainty given by the damping ratio variability on the Elastic response pseudo acceleration spectrum, the pseudo acceleration was calculated using three different values of damping ratio, $\xi - \sigma$ (i.e. 1.6%), ξ (i.e. 2.4%) and $\xi + \sigma$ (i.e. 3.2 %).

Results are illustrated in Figure 14. The pseudo acceleration peak on the top significantly varies from about 80 to about 120 m/s^2. At contrary, the variability of the model first natural frequency do not affect the seismic response because the corresponding natural period at prototype scale value remains largely, around 5 s.

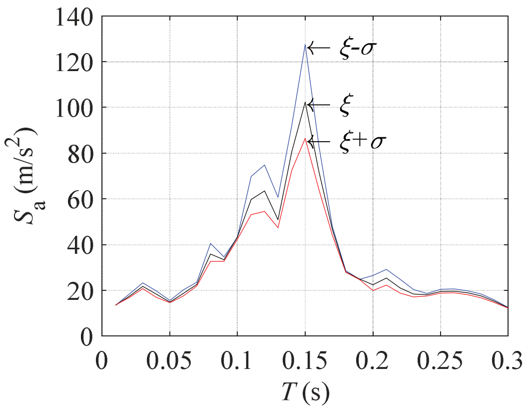


Figure 14. Damping ratio uncertainty propagation on the Elastic response pseudo acceleration spectrum on the top.

CONCLUSIONS

The design of in-scale model for multi-hazard experiments (i.e. seismic and wind–structure interaction) was discussed here. Structural elements of a prototype assumed as case of study were sized and subsequently scaled through aeroealstic scaling lows in order to design a model for dynamic wind tunnel tests. The same model was tested on a shaking table with a double purpose, firstly to compute the model dynamic identification (i.e. natural frequencies and damping) and secondly, to estimate the acceleration amplification.

This study is propaedeutic to multi-hazard experimental tests since it is able to investigate contemporarily wind action and seismic excitations at the operating limit state.

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