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Seismic vulnerability of a high-rise hospital building using field monitoring data

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***Abstract* – In this study, the seismic vulnerability of a high-rise hospital building is evaluated, combining numerical analysis and field monitoring data. The target building is an eight-storey reinforced concrete (RC) structure composed by two units separated through a structural joint. The assessment of the dynamic characteristics is performed using ambient noise measurements recorded by a temporary seismic network deployed inside the hospital. The modal identification results are used to update and better constrain the initial finite element model of the building, which is based on the available design and construction documentation plans. Three-dimensional incremental dynamic analysis is performed to derive the fragility curves for the initial as built model and for the real structures as they are nowadays. Results indicate that the consideration of the actual state of structures may significantly alter their expected seismic performance leading to higher vulnerability values.**

I. INTRODUCTION

Dynamic characterization of civil engineering structures (natural frequencies, damping ratios, mode shapes) becomes increasingly important in a wide range of research and application fields, such as dynamic response prediction [1], finite element updating [2], structural health monitoring and damage detection [3]. In this context the use of field monitoring data for identifying the actual state of the structures has drawn great attention in engineering community for developing real time assessment tools and reducing uncertainties involved in the risk assessment procedure [4].

In order to derive building-specific fragility curves based on field monitoring data (e.g. ambient noise measurements), which correspond to the actual state and vulnerability of a structure, the measured modal parameters can be used to "correct" the finite element models to better reflect the measured data than the initial ones. The lack of correlation between the numerical structural models and experimental observations may be attributed to poorly known boundary conditions, unknown material properties or modeling simplifications. These uncertainties cause the predicted dynamic response of a structure to be different from the measured dynamics of the real structure.

The present study aims at assessing the building-specific seismic vulnerability of one of the main buildings of the most important hospital in Thessaloniki (AHEPA) based on field monitoring data. This RC building has been selected as test site for the European funded REAKT project (http://www.reaktproject.eu/). "Building-specific" in this case means the present (actual) seismic vulnerability of the building considering all possible geometrical modifications, mass distributions and material deterioration. Ambient noise measurements are used to derive the experimental modal response of the hospital building and identify its dynamic properties performing Operational Modal Analysis (OMA). The modal identification results are then used to update and better constrain the initial finite element model of the building, which is based on the design and construction documentation plans provided by the Technical Services of the hospital in order to evaluate its real (actual) fragility and vulnerability. For both the initial and updated finite element models, incremental dynamic analysis (IDA) is performed for 15 real ground motions corresponding to the regional seismic hazard, in order to derive the fragility curves and evaluate the actual state of the hospital building.

II. DESCRIPTION OF AHEPA HOSPITAL AND INSTRUMENTATION ARRAY

The AHEPA general hospital in Thessaloniki is one of the largest hospitals in northern Greece, located in the campus of Aristotle University. It is a major teaching and research center and part of the National Healthcare System of Greece. The target building hosts both administration and hospitalization activities. It was constructed in 1971 and is considered representative of structures that have been designed according to the old 1959 Greek seismic code ('Royal Decree' of 1959), where the ductility and the dynamic features of the constructions are ignored. It is an eight storey infilled structure and its special feature is that it is composed of two adjacent tall building units that are separated through a structural joint (Fig. 1).

UNIT 1 covers a rectangular area of 29m by 16m while UNIT 2 has a trapezoidal area of 21 by 27 by 16m.

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Table 1. Characteristics of the adjacent hospital units.

RC building	Total mass (t)	f_c (MPa)	f_y (MPa)	f_m (MPa)
UNIT 1	3719	14	220	3
UNIT 2	3112	14	220	3

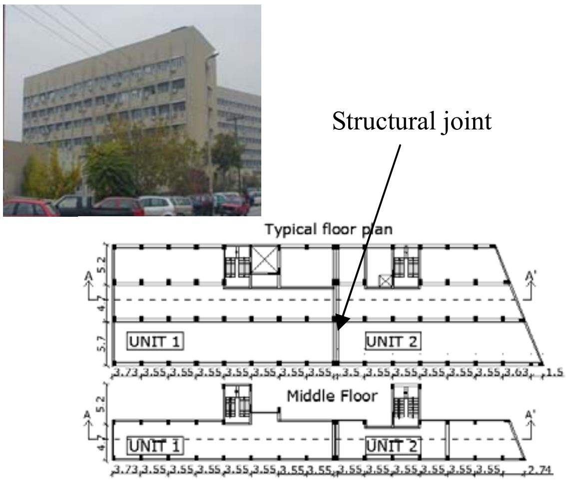

Fig. 1. Typical floor plan with the structural joint.

The total height of the building with respect to the foundation level is 28.6m with a constant inter-storey height of 3.4m except for the second floor where the height increases to 4.8m due to the presence of a middle floor level which covers only a part of the typical floor plan (Fig. 1). From the structural point of view the building's force resisting mechanism comprises of longitudinal and externally transverse reinforced concrete moment resisting frames (Fig. 1). The foundation of UNIT 1 consists of simple footings whereas in UNIT 2 the footings are partially combined with a raft foundation. Table 1 presents the main characteristics of the two units, namely the mass, the strength of concrete (f_c), steel (f_y) and masonry infill (f_m).

In February 2013, a temporary array of 36 triaxial seismometers (Mark Products L4C-3D of 1Hz coupled to EarthData Logger, PR6-24, 24 bit digitizers) was deployed in the hospital building under the responsibility of the Soil Dynamics and Geotechnical Earthquake Engineering of the Aristotle University of Thessaloniki (SDGEE-AUTH) and in close cooperation with Helmholtz Centre Potsdam, German Centre for Geosciences (GFZ). Each floor of the building was instrumented with four seismometers in order to ensure the observability of translational and torsional modes. Fig. 2 reports the location of the seismometers installed along the middle corridor near and far from the structural joint. North direction of the stations was placed parallel to the longitudinal structural direction of the building. Ambient noise was recorded simultaneously for 4 hours in all stations with a sampling rate of 500Hz and gain 10.

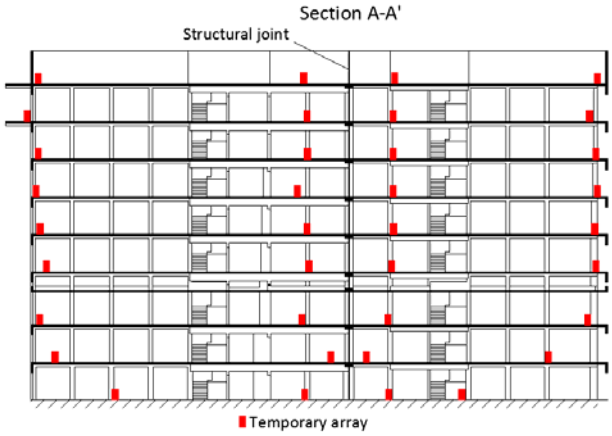

Fig. 2. Section A-A' along the longitudinal direction of the hospital building with the temporary instrumentation.

III. VULNERABILITY ASSESSMENT USING MONITORING DATA

A. Operational modal analysis

To evaluate the dynamic characteristics of the two hospital building units (UNIT 1 and UNIT 2), i.e. the natural frequencies and mode shapes, OMA is performed using MACEC 3.2 software [5], considering only the horizontal components of the noise records. The grid of the models was built so that the defined nodes correspond to nodes that have been actually measured. The stations that are used for the identification process are illustrated in Fig. 2. System identification and modal analysis of the structural models are conducted using non-parametric (Frequency Domain Decomposition-FDD [6]) and parametric (Stochastic Subspace Identification-SSI [7]) identification techniques. The results of the FDD and SSI analyses, namely the singular values and stabilization diagrams respectively, for the two adjacent buildings are presented in Fig. 3.

In Table 2, the eigenfrequencies computed with the two identification methods are summarized for both units. It is observed that the estimated frequency values for the five well separated modes are very close to each other (practically the same for the first three modes) for both of the applied methods, as well as for the different identified system models. The structures are exhibiting coupled sway and torsional modes in the frequency range of interest, which is expected in case of geometric and structural irregularities or eccentricities between the center of mass and center of rigidity. The highly coupled mode shapes confirm the complex vibrational characteristics of the structures especially for the first two identified frequencies. The resonant frequencies of the two adjacent units are very close, which may be attributed to their similar mass and stiffness properties.

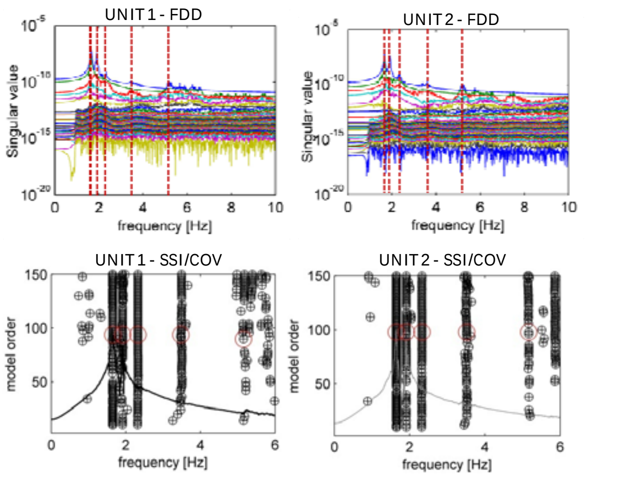

Fig. 3. Modal identification based on the FDD and the reference-driven SSI(-cov) methods using ambient noise measurements.

Table 2. Modal identification results for UNIT 1 and UNIT 2 estimated using the FDD and SSI identification techniques.

Mode	Mode Type	UNIT 1		UNIT 2	
		FDD (Hz)	SSI (Hz)	FDD (Hz)	SSI (Hz)
1	Coupled translational	1.65	1.65	1.65	1.65
2	Coupled translational	1.90	1.91	1.91	1.91
3	Torsional	2.33	2.33	2.35	2.33
4	1st longitudinal	3.50	3.47	3.58	3.52
5	2nd longitudinal	5.20	5.15	5.22	5.16

B. Finite element model updating

Model updating aims at the "correction" of the finite element model based on data obtained from measurements conducted on the test structure. The main purpose is to modify iteratively updating parameters to result in structural models that better reflect the measured data than the initial ones. The aim of the procedure is to conduct an extensive parametric study of the hospital buildings considering the variation in structural parameters (e.g. modulus of elasticity), investigating the sensitivity of the model to material properties, and how the latter may affect the overall stiffness of the structure.

The "initial" numerical model of the buildings under study is based on the design and construction documentation plans provided by the Technical Services of the hospital. The numerical modeling is conducted for the two adjacent units separately using OpenSees [8]. Elastic beam-column and truss elements are employed to model the linear RC elements (beams and columns) and masonry infills respectively. For the linear modeling of the masonry infills a double strut model is adopted to represent the in-plane behavior of the infill panel. Fixed base conditions are assumed for both structural models.

For the updating procedure, the compressive strength of the masonry infill f_m is selected as sensitivity parameter to take into account the uncertainties of the material behavior as well as the possible heterogeneity between the material properties of the different infill parts. A suite of numerical models is generated considering a proper distribution for f_m and defining possible scenarios adopting different infill masonry compressive strength values and configurations. The mean value of the masonry compressive strength μ=3MPa and its covariance COV=20% are defined based on a normal distribution according to [9]. The different values of compressive strength for the considered updating scenarios are subsequently computed based on the mean and standard deviation s according to the adopted normal distribution considering a limit range for the mean value of $\mu\text{-}3s \leq f_m \leq \mu+3s$. Then the elastic modulus in compression, which is used as input parameter to simulate the masonry infills, is estimated based on the adopted mean value for the compressive strength from $E_m=1000f_m$ [10]. Different scenarios are investigated regarding the variation in E_m and the considered configurations of the masonry infills for the selection of the 'best' model. Modal analyses for all the derived numerical models are performed in OpenSees for the three dimensional elastic linear finite element models of the two adjacent buildings separately (UNIT 1 and UNIT 2). Only one among them is considered as the 'best' model representing the observed dynamic response. The selection of the 'best' model is made based on the evaluation of the Modal Assurance Criterion (MAC) defined as:

$$MAC_{ij} = \frac{\left(\varphi_j^T \varphi_{Ei}\right)}{\left(\varphi_j^T \varphi_i\right)\left(\varphi_{Ei}^T \varphi_{Ei}\right)} \qquad (1)$$

where φ_j is the eigenvector j from numerical model and φ_{Ei} the eigenvector i from field monitoring test. A good correlation between the two tested modes is considered to be achieved for MAC values greater than 0.8. The scenario that represents most accurately the experimental results for the modes under investigation is the one shown in Fig. 4. The elastic moduli in compression of masonry infills adopted for this scenario were the following: E_{mlong1}= 3GPa (f_m=μ=3MPa), E_{mlong2}=1.8GPa (f_m=μ-2s=1.8MPa), $E_{mtransv1}$=3GPa (f_m=μ=3MPa) and $E_{mtransv2}$=4.8GPa (f_m=μ+3s=4.8MPa).

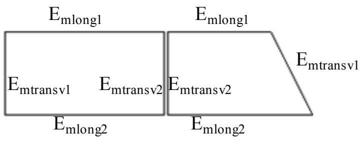

Fig. 4. The updating scenario that best reflects the experimental results.

Table 3. Comparison of the updated finite element model of UNIT 1 with the initial model and the experimental results.

Mode	Frequencies (Hz)			
	Initial	Updated	Experimental	MAC
1	1.46	1.56	1.65	0.96
2	2.06	1.89	1.91	0.94
3	2.70	2.70	2.33	0.97

Table 4. Comparison of the updated finite element model of UNIT 2 with the initial model and the experimental results.

Mode	Frequencies (Hz)			
	Initial	Updated	Experimental	MAC
1	1.50	1.54	1.65	0.98
2	2.05	1.89	1.91	0.45
3	2.77	2.86	2.33	0.94

Due to the complexity of the structure under study only the first three modes are considered in the updating process, which represent the fundamental deformation modes of the structure and activate approximately 80% of the total mass of the building units. In Tables 3 and 4 the results of the updating methodology for UNIT 1 and UNIT 2 are presented respectively.

The eigenfrequencies and mode shapes of the updated finite element models are compared to the initial ones as well as to the experimental results. It is seen that for UNIT 1, the updated finite element model correlates well with the experimental results for all the modes under investigation (MAC>0.8). For UNIT 2 on the other hand, MAC values are high for the 1^{st} and 3^{rd} mode, indicating the satisfactory correlation between analytically and experimentally calculated modal parameters, whereas for the 2^{nd} mode it was not possible to achieve MAC values greater than 0.8. This may be attributed to the fact that the structural configuration of UNIT 2 (trapezoidal plan section) did not allow to capture the 2^{nd} mode shape and probably another sensitivity parameter related not only to the structural stiffness (such as the masonry compressive strength) but also to the storey mass or a combination of several parameters related to both stiffness and mass, would be more appropriate. Given however the difficulties in proper asserting the mass properties of the complex hospital building (e.g. distribution of mass along height and floor), this parameter is not used in the updating procedure as the associated uncertainties may reduce the accuracy of the results.

C. Inelastic finite element modeling

The numerical modeling of the structure is conducted in OpenSees [8]. Inelastic force-based formulations are employed for the simulation of the nonlinear three-dimensional beam-column frame elements. The applied formulations allow both geometric and material nonlinearities to be captured. Distributed material plasticity along the element length is considered based on the fiber approach to represent the cross-sectional behavior. The Popovics [11] concrete model is used to define the behavior of the concrete fibers, yet different material parameters are adopted for the confined (core) and the unconfined (cover) concrete. The steel reinforcement is modeled using the uniaxial 'Steel01' material to represent a uniaxial bilinear steel material with kinematic hardening described by a nonlinear evolution equation. For the nonlinear modeling of the masonry infills inelastic struts are used to represent infill walls. Each strut is assigned an elasto-plastic force displacement relationship representing initial stiffness and peak strength behavior of the masonry. For both structural models fixed base conditions are assumed.

D. Seismic input motion

The selected scenario earthquake consists of a set of 15 real ground motion records obtained from the European Strong-Motion Database (http://www.isesd.hi.is). They are all referring to stiff soil conditions classified as soil type B according to EC8 with moment magnitude (M_w) and epicentral distance (R) that range between $5.8<M_w<7.2$ and 0<R<45km respectively. The primary selection criterion was the average acceleration spectra of the set to be of minimal "epsilon" [12] at the period range of 0.00<T<2.00sec with respect to the acceleration spectrum adopted from SHARE for a *475 year* return period (http://portal.share-eu.org:8080/jetspeed/portal/). The optimization procedure was performed making use of the REXEL software [13]. Fig. 5 shows the mean normalized elastic response spectrum of the records in comparison with the corresponding reference spectrum adopted from SHARE. A good match between the two spectra is achieved.

E. Incremental dynamic analysis

The IDA procedure [14] is used to determine the seismic performance and assess the seismic vulnerability of the initial and updated finite element models of UNIT 1 and UNIT 2.

Within this study the damage measure is expressed in

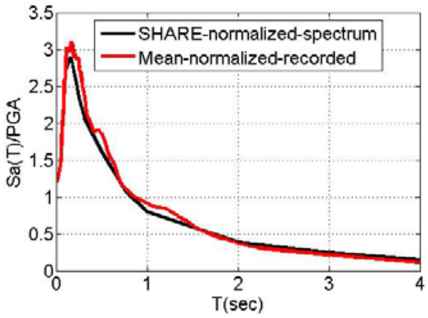

Fig. 5. Normalized average elastic response spectrum of the input motions compared with the corresponding reference spectrum adopted from SHARE.

terms of maximum inter-storey drift ratio, maxISD. More specifically the maximum peak SRSS drift (i.e. the maximum over all stories of the peak of the square-root-sum-of-squares of each storey's drift) in the two principal directions is selected [15]. The seismic intensity is described using peak ground acceleration (PGA) recorded on rock outcropping or soil type A according to EC8.

IDA is conducted for the structural models by applying the 15 progressively scaled records. By interpolating the derived pairs of PGA and maxISD for each individual record, 15 continuous IDA curves for each structural model are derived. For the purpose of the present study, two limit states are defined in terms of maximum inter-story drift ratio, maxISD, representing the immediate occupancy (IO) and collapse or near collapse prevention (CP) performance levels. The first limit state, namely the Immediate Occupancy corresponds to the yielding point where the elastic branch gives place to the post-elastic branch. The second limit state is assigned at a point where the IDA curve is softening towards the flat line, but at low enough values of maxISD so that we still trust the structural model. Thus different IO and CP limit state values are chosen on the IDA curves for the same structure depending on the reference finite element model (initial or updated) and the individual record. For both initial and updated models of UNIT 1 and UNIT 2 the median of the first limit state is found equal to 0.1% while the median of the defined CP limit states in terms of SRSS inter-storey drift (maxISD) is found to be equal to 1.4% and 1.1% for the initial and updated models respectively.

F. Derivation of fragility curves

A fragility curve shows graphically the relationship of the probability of exceeding a predefined level of damage (e.g. IO, CP) under a seismic excitation of a given intensity. The results of the IDA (PGA - maxISD values) are used to derive the fragility curves for both analyzed buildings, expressed as a two-parameter lognormal cumulative distribution function:

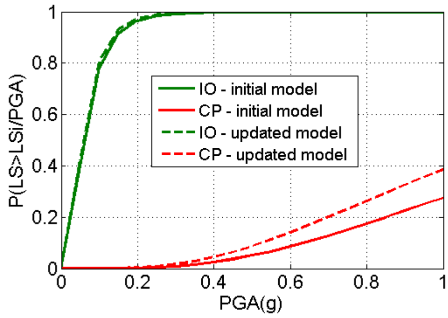

Fig. 6. Comparative plot of the fragility curves derived for the initial and updated models of UNIT 1

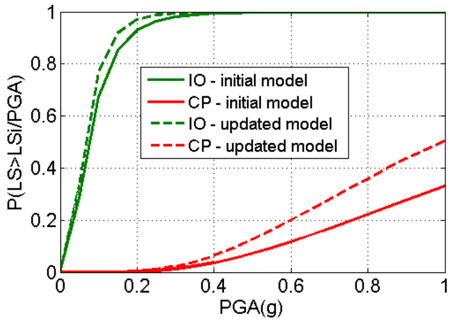

Fig. 7. Comparative plot of the fragility curves derived for the initial and updated models of UNIT 2.

$$P[DS/IM] = \Phi\left(\frac{\ln(IM) - \ln(\overline{IM})}{\beta}\right) \qquad (2)$$

where Φ is the standard normal cumulative distribution function, IM is the intensity measure of the earthquake expressed in terms of PGA (in units of *g*), $\overline{IM}$ and β are the median values (in units of *g*) and log-standard deviations respectively of the building fragilities and *DS* is the damage state. The median values of PGA corresponding to the prescribed performance levels are determined based a linear regression fit of the logarithms of the PGA- maxISD data which minimizes the regression residuals. In the present study the uncertainty associated with the demand is taken into consideration by calculating the dispersion of the logarithms of PGA - maxISD simulated data with respect to the regression fit. The log-standard deviation value in the capacity is assumed to be 0.3 for the low code structures following the HAZUS prescriptions [16].

Fragility curves are derived for the initial and updated finite element models of UNIT 1 and UNIT 2. The initial "as built" numerical models are based on the available design plans and correspond to the initial state of the structures, whereas the updated models reflect the measured responses and therefore represent their actual state. The "real-time" fragility curves are obtained by assessing the seismic performance of the updated numerical models and are compared with the initial ones in Fig. 6 and 7 for UNIT 1 and UNIT 2 respectively. It is observed that the updated curves present a shift to the left in comparison to the initial ones, indicating an increase in the structures vulnerability which is more noticeable for the CP limit state and for large intensities.

IV. CONCLUSIONS

The "actual" seismic vulnerability of one of the main buildings of the most important hospital in Thessaloniki (AHEPA) was assessed based on field monitoring data. The special feature of the target building is that it is composed of two adjacent tall units that are connected with a structural joint.

The modal identification results were used to update and better constrain the "initial" finite element models of the two adjacent units. Incremental dynamic analysis was performed for the initial and updated structural models to evaluate the seismic performance of the buildings when their actual state is taken into account.

The fragility functions were derived for the IO and CP limit states in terms of PGA for both structural units. An overall increase in structures fragility for the updated models was observed in comparison to the ones corresponding to their initial state which are currently used so far. Thus the present study provides further insight on the assessment of the "real-time" seismic vulnerability of typical RC buildings using field monitoring data, taking into account the actual state of the structure (degradation due to time, possible pre-existing damages, changes in geometry and mass distribution, etc). The proposed updating methodology can be used to yield more reliable structural models with respect to their real conditions in terms of structural detailing, mass distribution and material properties. Furthermore the proposed methodology can be extended for "real-time" risk assessment and post-seismic fragility updating.

V. ACKNOWLEDGMENTS

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