BUILDING – SPECIFIC VULNERABILITY ASSESSMENT OF CRITICAL BUILDINGS USING SHORT TERM FIELD MONITORING DATA

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ABSTRACT

In the context of seismic vulnerability assessment of reinforced concrete (RC) buildings, the use of field monitoring data constitutes a significant tool for the representation of the actual structural state, reducing uncertainties associated with the building configuration properties as well as many non-physical parameters (age, maintenance, etc.), enhancing thus the reliability in the risk assessment procedure. In this study, the seismic vulnerability of existing RC buildings is evaluated, combining through a comprehensive methodology, the numerical analysis and field monitoring data. The proposed methodology is highlighted through the derivation of building specific fragility curves for a nine-storey RC structure that has been designed with old seismic design codes. The assessment of the dynamic characteristics is performed using ambient noise measurements recorded by a temporary seismic network which was deployed inside the building. 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 real structure as it is nowadays. Fragility curves are derived in terms of outcropping peak ground acceleration (PGA) for the Immediate Occupancy (IO) and Collapse Prevention (CP) limit states for the real structure.

Keywords: Building monitoring; Operational modal analysis; Finite element model updating; Building-specific fragility curves

1. INTRODUCTION

Seismic fragility curves are commonly used for assessment of structural performances. One of the major challenges is the incorporation of the various uncertainties that are involved in the assessment procedures with an adequate level of accuracy. There are several studies in the literature which have addressed the assessment of seismic safety of buildings using nonlinear analyses procedures to assess the collapse risk of buildings and to quantify the involved modeling uncertainties (e.g. Haselton et al. 2011, Liel et al. 2011, Gokkaya et al. 2016, Kosič et. al 2016).

In the context of seismic vulnerability assessment of buildings, the use of field monitoring data constitutes a significant tool for the representation of the actual structural state, reducing uncertainties associated with the building configuration properties as well as many non-physical parameters (age, maintenance, etc.), enhancing thus the reliability in the risk assessment procedure. The present work aims at the evaluation of the seismic vulnerability of existing RC buildings combining through a comprehensive methodology, the numerical analysis and field monitoring data. The target structure

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under study is the Administration building from the Aristotle University of Thessaloniki which is
designed with Greek low seismic code provisions.
The following sections describe in detail the building under investigation as well as the different steps
of the applied methodological framework, starting from the instrumentation layouts implemented on the
buildings and resulting to their nonlinear seismic response evaluation and vulnerability assessment.

2. APPLICATION STUDY

2.1 Structural description

The Aristotle University of Thessaloniki (AUTh) is the largest University in Greece. The main campus
is located in the centre of the city of Thessaloniki and most of the buildings hosting the faculties and
schools were built before 1985 and are classified as low seismic code structures.
The Administration building is a critical building concentrating the most important administrative and
economic activities of the University (Figure 1). It was constructed in 1964 and is considered
representative of structures that have been designed according to the old 1959 Greek seismic code, where
the ductility and the dynamic features of the constructions are ignored. The design seismic PGA
according to the particular code for Thessaloniki is defined at 0.06g which is a much lower value
compared to the present code (i.e. 0.16g). During the 1978 Thessaloniki earthquake (M=6.5, R=26.7
km), which generally caused extensive damages and the collapse of one high-rise residence structure,
the Administration building suffered moderate damages and was subjected to local repairs such as filling
of the concrete cracks with epoxies and reinforcement of specific beam elements with additional rebars
and shotcrete concrete.

It is a nine-storey structure with basement and presents irregularities in both elevation and plan as shown
in Figure 2 and 3. In particular, while the basement and the ground floor level of the structure cover a
rectangular area of 56.60 m by 21.60 m and 59.50 m by 25.50 m respectively, the typical floor plan has
a rectangular cross section of 44.80 m by 10.80 m. The total height of the building with respect to the
foundation level is 34.20 m with the interstorey height varying along the building elevation. More
specifically the height of the basement and ground floor is equal to 4.60 m and 4.00 m respectively
whereas the typical floor is 3.20 m high. From the structural point of view the building is characterized
by a dual force resisting mechanism comprising frames as well as core walls.

Due to the eccentricities existing between the center of mass and the center of rigidity, the building is
expected to exhibit torsional effects. Furthermore it should be noted that in the basement and ground
floor levels there are structural joints connecting the beam with the column elements and their location
are shown in Figure 2 and 3. Finally it should be noted that the building is connected with nearby
structures. This connection is however limited only at the 1st storey level.

Using the SYNER-G taxonomy (Pitilakis et al. 2014) for RC structures to describe the typology of the
Administration building, it may be defined as high-rise dual system designed based on low seismic code
provisions.
In the fall of 2015, ambient noise measurements were performed at the campus to assess the soil
conditions and site effect characteristics namely the soil characterization in terms of resonant frequency,
amplification factor and shear wave velocities with depth close to the building under study. The results
showed that the foundation subsoil conditions of the area of the building and below the first 8-9 m thick
surface layer (composed mainly by artificial landfills), is characterized by quite stiff soils with Vs
velocities greater than 400-500 m/sec. Based on these results the foundation soil at the two buildings
can be characterized as soil type B according to EC8 soil classification. The seismic bedrock (Vs > 1000
m/s) is found below 35 m depth.

2.2 Instrumentation array

A temporary instrumentation array was implemented at the end of September 2015 under the
responsibility of the Soil Dynamics and Geotechnical Earthquake Engineering of the Aristotle
University of Thessaloniki (SDGEE-AUTH) and in close cooperation with the Technische Universität
Berlin (TU-Berlin) and the Helmholtz Centre Potsdam, German Centre for Geosciences (GFZ). Ambient
noise measurements are used for the dynamic characterization of the building deploying a dense network of stations equipped with velocimeters. The instrumentation layout included 38 triaxial geophones of 4.5 Hz natural frequency connected to CUBE digitizers. GPS antennas guaranteed the time synchronizations among all instruments. The sensors recorded along the two orthogonal horizontal and along the vertical directions (three components). The two horizontal components are oriented along the longitudinal and transversal direction of the building. Ambient noise was recorded simultaneously for about 20 hours in all stations with a sampling rate of 400 Hz. In order to capture the translational and torsional modes of the building, 4 sensors were installed at the corners of each floor close to the vertical structural elements (i.e. RC columns). Figure 4 illustrates the location of the sensors of the temporary network inside the building.

Figure 1. Administration building at AUTh campus

Figure 2. Plan view of the basement, the ground floor and the typical floor of the Administration building
4. MODAL ANALYSIS AND FINITE ELEMENT MODEL UPDATING

4.1 Experimental modal analysis

To evaluate the dynamic characteristics of the Administration building, namely the natural frequencies and mode shapes, system identification and Operational Modal Analysis (OMA) were performed using MACEC 3.2 software (Reynders et al. 2011). Operational modal analysis is performed considering only the horizontal components of the measurements. The grid of the model is built so that the defined nodes correspond to the nodes that have been actually measured. The time window used for OMA has duration of 1800 sec (30 min) as tests for stability of the results showed that 30 minutes are enough to get reliable results. Before identification, the data were decimated with a factor of 10 and filtered with a low-pass anti-aliasing filter with a cut-off frequency of 20 Hz and re-sampled at 40 Hz reducing thus the number of data avoiding unnecessary
computational burden in the modal analysis where the frequency of interest are smaller than 20 Hz.

In order to verify and enhance the modal identification results, analyses have been conducted using both non-parametric and parametric identification techniques.

Modal analysis of the identified non-parametric models has been conducted based on the Frequency Domain Decomposition (FDD) method. Modal identification based on parametric methods was conducted by applying the reference-based covariance-driven stochastic subspace method (SSI).

The results of the nonparametric and parametric analyses for the Administration building are presented in Figure 5. Figure 6 shows representative eigenfrequency and mode shape results of the identified modes based on the SSI analysis results. The building is exhibiting coupled sway and torsional modes in the frequency range of interest, which are expected in case of geometric and structural irregularities or eccentricities between the center of mass and center of rigidity. The highly coupled obtained mode shapes reveal the complex vibrational characteristics of the building especially for the first two identified frequencies. Although coupled, the predominant motion of the first mode is mainly along the transverse direction whereas the second one along the longitudinal direction.

**Figure 5. Identification results through OMA for the Administration building based on (a) FDD and (b) SSI methods**

**Figure 6. Eigenperiods/Eigenfrequencies and mode shapes for the first three identified modes for the Administration building (T: period, f: frequency)**

### 4.2 Comparison between the analytical and the experimental modal analysis results. Finite element model updating

The main purpose of this task is to modify iteratively updating parameters to result in a structural model of the Administration building that will better reflect the measured data than the initial one which was
built based on the design and construction plans. The initial elastic model is constructed in SeismoStruct (SeismoSoft, v.7) using beam-column elements for the wall and constraints for the diaphragm modeling respectively. The frame elements were simulated as linear elastic beam-column elements. The support conditions of the structure were considered as fixed base. In order to take into account the existence of the basement, the translational degrees of freedom at the ground floor level were restrained. Finally link elements were utilized for the structural joint modeling at the basement and ground floor level. Link elements were also used to model the connection of the building with the nearby structures at the 1st storey level. The total mass of the structure is estimated equal to 7219 tn taking into account the self-weight of the structural elements as well as the dead and live load acting on each floor level. Based on the available data, the strength class of concrete and reinforcement steel are considered as B300 (C25/30) and StIIIb (S400) respectively.

If not serious geometrical modifications are identified, as in the present case, structural features, such as material or mass properties, are likely to be selected as updating parameters in order to increase the correlation between the observed dynamic response of the structure and the predicted from the numerical modal model. In this particular case as updating parameter the stiffness parameter of the link elements $K_0$ is selected since no data are available for this property and therefore its definition includes high uncertainty levels. More specifically six stiffness parameter values that correspond to the translational and rotational degrees of freedom are investigated.

The updating is performed not only to improve the frequencies of the considered modes of the initial numerical model but also to calibrate the numerical mode shapes in order to fit the experimental data. A manual updating scheme is applied generating a suite of numerical modal models considering different values of the stiffness parameter $K_0$. Modal analyses are performed for all the derived numerical models. Only one among them was judged as superior compared to the initial model considered at the beginning of the updating process; this one is characterized as the ‘best’ model representing the observed dynamic response based on the noise measurements.

The selection of the ‘best’ model is made based on the evaluation of the Modal Assurance Criterion (MAC, Allemang and Brown 1982). The computation of the MAC values and the correlation of the responses between the experimental and numerical modal model are made at the measured nodes for which actual recorded noise data are available. A good correlation between the two tested modes was considered to be achieved for MAC values greater than 0.8. The updating scenario that was found to represent most accurately the experimental results for the modes under investigation considers the following stiffness parameter values: $K_0=10^4$ kN/m for the translational and $K_0=10^6$ kNm/rad for the rotational degrees of freedom.

Figure 7 shows the comparison between the updated numerical and experimental modal models of the Administration building in terms of resonance periods and mode shapes presenting also the resulting MAC values for the three first modes. The eigenfrequencies and mode shapes of the updated finite element model are compared to the initial ones as well as to the experimental results. It is seen that the updated model correlates well with the experimental results for all the modes under investigation (MAC > 0.8).

5. DERIVATION OF BUILDING – SPECIFIC FRAGILITY CURVES

5.1 Nonlinear finite element modeling

The nonlinear numerical modeling of the updated structure is conducted using SeismoStruct. Inelastic force-based formulations are employed for the simulation of the nonlinear three-dimensional, with six degrees of freedom, beam-column 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. Each fiber is associated with a uniaxial stress-strain relationship; the sectional stress-strain state of the beam column elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibers in which the section is subdivided. It should be noted herein that considering fiber modeling for the representation of the members’ nonlinearity, phenomena typical of RC elements such as bond slip and bar pullout are not taken into account. The concrete model of Mander et al. (1988) is used to define the
behavior of the concrete fibers while for the steel reinforcement the bilinear steel model with kinematic strain hardening was implemented. Tables 1 and 2 summarize the material properties of the defined constitutive laws. Additionally, threshold values are defined in SeismoStruct in terms of reinforcement steel strain values in order to identify the instants at which different performance criteria are reached.

Table 1. Properties of the defined concrete material model.

<table>
<thead>
<tr>
<th>Modulus of Elasticity (GPa)</th>
<th>Mean compressive strength (MPa)</th>
<th>Mean tensile strength (MPa)</th>
<th>Strain at peak stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>31</td>
<td>24</td>
<td>0.002</td>
</tr>
</tbody>
</table>

Table 2. Properties of the defined steel material model.

<table>
<thead>
<tr>
<th>Modulus of Elasticity (GPa)</th>
<th>Yield strength (MPa)</th>
<th>Strain hardening parameter</th>
<th>Fracture / buckling strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>400</td>
<td>0.005</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Figure 7. Comparison of the updated finite element model of the Administration building and the experimental results (T: period, f: frequency)
5.2 Selection of the input motion

To perform the nonlinear dynamic analysis of the building a target spectrum for stiff soil conditions ($V_{s,30}=410$ m/s, Pitilakis et al. 2016) corresponding to the normal seismic design scenario (i.e. $T_m=475$ years) and a suite of acceleration time histories are needed representative of this scenario. The target spectrum is defined based on the disaggregation of the probabilistic seismic hazard analysis (PSHA) results for the Aristotle University area (SRM-LIFE project, coordinator Prof. K. Pitilakis). This study has shown that the most significant contribution to seismic hazard is associated with the Anthemountas fault system (i.e. a normal fault) regardless of the return period. In particular, for the 475 years scenario, it was shown that the fault yields the maximum annual exceedance probability for a certain PGA value with a moment magnitude $M_w$ of 5.675 and an epicentral distance $R_{epi}$ of 11.67 km. The $R_{jb}$ distance (i.e. the closest distance to the surface projection of the rupture zone) and the rupture distance $R_{rup}$ (i.e. the closest distance to rupture zone) are estimated as 5 km and 10 km respectively. It was shown that the median spectra plus 0.5 standard deviations provided by Akkar et al. (2014) describes adequately the hazard in the study area for the 475 years scenario. Therefore it is used as the target spectrum for the selection of the seismic records.

A representative set of 20 accelerograms from 10 stations (10x2 directional components of ground motion) was selected from the European Strong-Motion Database. They are all referring to stiff soils (soil type B according to EC8) with moment magnitude ($M_w$) and epicentral distance that range between $5.5<M_w<6.5$ and $0<R<45$km respectively. The primary selection criterion is the average acceleration spectra of the set to be of minimal “epsilon” (Baker and Cornell 2005) at the period range of $0.00<T<2.00$sec with respect to the corresponding 5% damped median plus 0.5 standard deviations Akkar et al. (2014) spectrum. The epsilon of a given Intensity Measure IM is defined as the ratio of recorded minus predicted IM to the standard deviation. Thus the selection of peculiar features in the records that could bias the structural response and consequently the fragility is avoided. The optimization procedure is performed by making use of the REXEL software (Iervolino et al. 2010) that allows obtaining combinations of accelerograms, which on average are compatible to the reference spectrum. Figure 8 depicts the mean elastic response spectrum of the records in comparison with the corresponding median plus 0.5 standard deviations Akkar et al. (2014) spectrum. As shown in the figure, a good match between the two spectra is achieved.

![Figure 8. Mean elastic response spectrum of the input motions in comparison with the corresponding reference spectrum proposed by Akkar et al. (2014) plus 0.5 standard deviation](image)

5.3 Incremental dynamic analysis

The incremental dynamic analysis (IDA) procedure (Vamvatsikos and Cornell 2002) is used to determine the seismic performance and assess the seismic vulnerability of the initial and updated finite element model of the Administration building. Within this study the damage measure is expressed in terms of maximum interstorey drift ratio. More specifically, the maximum peak SRSS drift, maxISD (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 (Wen and Song 2002). The seismic intensity is described using
peak ground acceleration (PGA) recorded on soil type B according to EC8.

IDA is conducted for the structural model of the Administration building by applying the 10 progressively scaled records (in both longitudinal and transverse direction). In particular, a tracing algorithm is applied for each record with an initial step of 0.1g, a step increment of 0.1g and a first elastic run at 0.05g. For certain records it was necessary to reduce the step size of the algorithm to increase the accuracy close to the flatline of the IDA curve. The minimum number of converging runs is allowed to vary from 8 to 10 per record depending on the characteristics of the structure and the records itself.

It should be noted at this point that for the IDA the connection of the building to the nearby structures is taken into account only for analyses cases that consider low seismic intensity for which the structure’s behavior is limited in the linear elastic branch. The limit intensity level where the interaction between the structures is expected to be weak was defined at 0.1g. Beyond that level the structures are expected to experience strong nonlinearities with progressive increase of the PGA and therefore no connection was considered, as their interaction may be completely different affecting consequently their seismic response and vulnerability. The fiber based approach that has been adopted for the nonlinear modeling of the structures simulates sideways collapse associated with strength and stiffness degradation along the total length of beams and columns. The analysis model does not directly capture column shear failure as the columns in this study are expected to yield first in flexure rather than experiencing direct shear failure, as in the case of squat non-ductile RC columns. The collapse modes are expected to be related to column flexural failures in the lower storeys which are defined for each seismic record based on the intensity of the input ground motion that results in structural collapse, identified in the analysis by excessive interstorey drifts. Damage at the core walls is expected to be limited and located mainly at the lower storey levels where the concentration of seismic load is higher. Furthermore, the torsional effects induced due to the eccentric location of the core walls in plan, lead to extensive damage distribution of the elements at the left side of the building, which is located far from the core walls. Finally partial collapse of the first floor is observed which can be attributed to the absence of frontal walls at this storey level causing significant stiffness irregularity in elevation and thus high inelastic deformation demands.

By interpolating the derived pairs of PGA and maxISD for each record 10 continuous IDA curves are derived. For the purpose of the present study, two limit states are defined on the IDA curve in terms of interstorey drift ratio based on the results and engineering judgment, representing the immediate occupancy (IO) and collapse or near collapse prevention (CP) performance levels. The first limit state, namely the IO performance level, corresponds to the point where the elastic behavior of the structure gives place to its post-elastic state. The second limit state is assigned at a point where the IDA curve is softening towards the flat line. The flatline indicates that Global Instability of the structure has been reached; therefore the CP thresholds are defined at low enough values of maxISD before reaching the flatline, so that we still trust the structural model (Vamvatsikos and Cornell 2004). Thus different IO and CP limit state values are chosen on the IDA curves for the same structure depending on the individual record. The medians of the defined IO and CP limit values in terms of SRSS maxISD are used to define the IO and CP limit states and are found to be equal to 0.2% and 1.9% respectively. The assignments of the IO and CP limit state points on the IDA curves corresponding to each record are shown in Figure 9. The dispersion that is observed may be attributed on one hand to the record-to-record variability in terms of frequency content and duration and on the other hand to the fact that PGA is used as intensity measure as in this case the seismic response and vulnerability depends on the input ground motion sets.

5.4 Derivation of building – specific fragility curves

The results of the IDA (PGA-maxISD) are used to derive the fragility curves for the Administration building, expressed as a two-parameter lognormal distribution function. Equation 1 represents the cumulative probability of exceeding a damage state DS conditioned on a measure of the seismic intensity IM:

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

\(\Phi\) is the cumulative standard normal distribution function.
where, $\Phi$ is the standard normal cumulative distribution function, IM is the intensity measure of the earthquake expressed in terms of PGA (in units of g), $\bar{IM}$ and $\beta$ are the median values (in units of g) and log-standards 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 on a regression analysis of the nonlinear IDA results (PGA-maxISD pairs). More specifically a linear regression fit of the logarithms of the PGA–maxISD which minimizes the regression residuals is adopted in the analysis cases. Figure 10 presents the PGA–maxISD relationships for the updated structural model of the Administration building.

Three primary sources of uncertainty are generally taken into account for the estimation of the total variability associated to each damage state, namely the variability associated to each damage state, the capacity of the structure and the seismic demand. 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 (NIBS, 2004). As far as the uncertainty in the definition of the damage states is concerned, the damage limit values are defined on the IDA curves and since they are considered building-specific, the additional uncertainty related to the definition of the damage states is taken into consideration through the dispersion of the defined limit.
values of IO and CP damage states. In particular, the corresponding $\beta$ is estimated 0.19 and 0.4 for the IO and CP damage states respectively. Under the assumption that the log-standard deviation components are statistically independent, the total log-standard deviation is estimated as the root of the sum of the squares of the component dispersions. The herein computed log-standard deviation $\beta$ values of the curves are equal to 0.56 and 0.66 for the IO and CP damage states respectively.

Figure 11 presents the building-specific fragility functions derived for the updated structural model of the Administration building while Table 3 summarizes the corresponding fragility parameters. For an example intensity level equal to 0.4g it is observed that the probability of the structure to experience slight damage is 100% which is expected as the IDA results showed already that the building develops relatively fast inelastic deformations due to its structural characteristics (structural eccentricities in plan and elevation). On the other hand the probability for the building to collapse under the considered intensity level is estimated approximately equal to 15%.

Table 3. Parameters of the derived fragility curves for the updated finite element model of the Administration building.

<table>
<thead>
<tr>
<th>Damage state</th>
<th>Median (g)</th>
<th>Dispersion $\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Occupancy (IO)</td>
<td>0.05</td>
<td>0.56</td>
</tr>
<tr>
<td>Collapse Prevention (CP)</td>
<td>0.80</td>
<td>0.66</td>
</tr>
</tbody>
</table>

Figure 11. PGA – maxISD relationships for updated finite element model of the Administration building

6. CONCLUSIONS

Ambient noise measurements have been employed to assess the real present seismic vulnerability of the Administration building of Aristotle University in Thessaloniki. The monitoring data were used to derive the experimental modal model of the building and to identify its modal properties based on system identification and OMA respectively.

The modal identification results were used to update and better constrain the initial finite element model of the structure under study, which was based on the design and construction documentation plans provided by the Technical Service of the University. A sensitivity parameter related to the structural joint stiffness properties was adopted for the updating procedure.

For the updated model of the Administration building three-dimensional IDA was performed to evaluate its seismic performance and to assess its vulnerability. The fragility functions were derived for the IO and the CP limit states in terms of PGA. Future work should present a comparison between the derived building-specific fragility curves with conventional generic curves adopted from the literature. Since those curves constitute crucial components in the framework of risk mitigation strategies (e.g. seismic safety and rehabilitation costs), it is of high importance to investigate whether they can be used for accurate fragility and loss estimates in the case of individual building assessment.
7. ACKNOWLEDGMENTS

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8. REFERENCES


