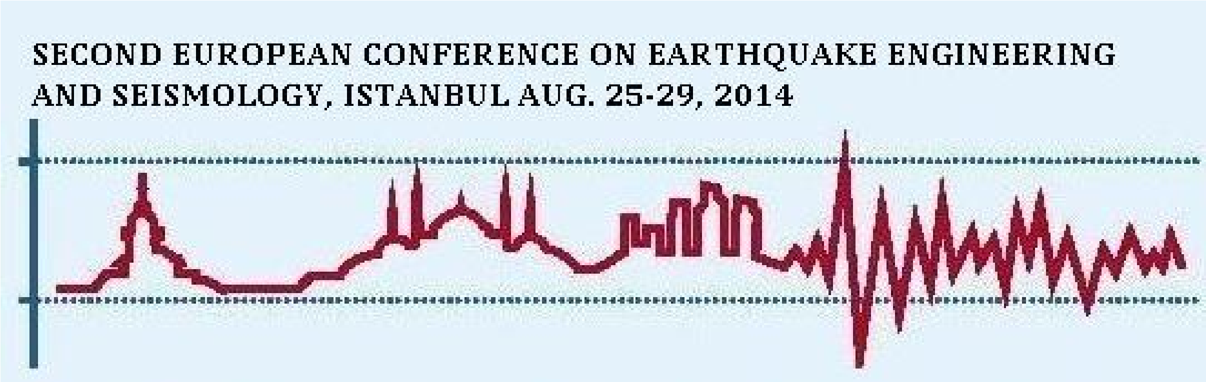


ANALYTICAL VULNERABILITY ASSESSMENT OF MODERN HIGHRISE RC MOMENT-RESISTING FRAME BUILDINGS IN THE WESTERN USA FOR THE GLOBAL EARTHQUAKE MODEL

Athanasia KAZANTZI[1], Dimitrios VAMVATSIKOS[2], Keith PORTER[3] and In Ho CHO[4]

ABSTRACT

The Global Earthquake Model (GEM) has commissioned the preparation of analytical seismic vulnerability guidelines for loss assessment of highrise buildings. The guidelines attempt to provide a practical method to assess the relationship between ground shaking and repair cost for a class of buildings, in face of the time, skill and computational challenges posed by a class rather than a single structure. An example is presented that reflects a class of modern (post-1980) highrise reinforced-concrete moment-frame (RCMRF) office buildings in the western United States. Seven characteristic “index” buildings were selected through moment-matching, so as to represent all important aspects of the class. Each is modeled and analyzed through Incremental Dynamic Analysis (IDA). To remove issues of sufficiency without performing record selection, an extensive comparative study of intensity measures (IMs) was undertaken. Scalar IMs, defined as the geometric mean of spectral acceleration values at adjacent periods, were found to be satisfactory. Then, for each of the seven index buildings, poor, typical, and superior-quality variants were considered to establish the seismic loss and its ratio over replacement cost new in each case. The end product is a high quality set of vulnerability curves, whose weighted moments are taken as the uncertain vulnerability function of the class.

INTRODUCTION

The Global Earthquake Model (GEM) is a grand effort to proffer a comprehensive open-source tool for large scale loss assessment studies. Given the lack of sufficient historical data on the seismic performance of a broad range of building classes worldwide, the value of an analytical model to perform vulnerability and, consequently, loss assessment becomes apparent. To this end, a set of guidelines was recently developed by Porter *et al*. (2014) aiming to offer a practical analytical method for assessing the relationship between the ground shaking and the repair cost for a building class. A class of buildings is often defined by structural material, lateral force resisting system, height range, and era of construction for a specific country or region. This is typically represented by a set of index structures (Reitherman and Cobeen, 2003) that are appropriately selected so as their key features reflect the most influential attributes vulnerability-wise whereas, at the same time their probabilistic combination approximates the distribution of the key features within the asset class.

The test bed for the present work is a set of highrise reinforced-concrete moment-resisting frames (RCMRFs), designed in accordance with the US regulations for high seismicity regions (mainly Western USA). By considering the three top influential attributes, the general properties of the

[1] Research Engineer, J.B. ATE, Herakleion, Crete, kazantzi@mail.ntua.gr
[2] Lecturer, National Technical University of Athens, Athens, divamva@mail.ntua.gr
[3] Research Professor, University of Colorado at Boulder, Boulder, keith.porter@colorado.edu
[4] Willis Research Fellow, University of Colorado at Boulder, Boulder, inho.cho@colorado.edu

index buildings are selected through moment-matching (Ching *et al*, 2009) to approximate the corresponding distribution of these characteristics in a highrise RC building population in California (Porter and Cho, 2013). These features are used to depict and accordingly modify a set of suitable RCMRFs from a database developed by Haselton *et al*. (2011).

Evidently, compared to a single building vulnerability assessment, the assessment of a building class poses additional time, skill and computational challenges that need to be clearly addressed and efficiently tackled. Furthermore, the proposed approach is constrained by the desire to (a) remain as close as possible to the current state-of-the-art, that is the 2nd generation performance based earthquake engineering represented by FEMA P-58 (ATC, 2012) (b) be accessible to any structural engineer with a master's degree (c) incorporate at least the most important uncertainties and (d) require either hours or days of analysis rather than weeks or months. Further constraints are imposed to the present problem by the nature of the highrise buildings, typically precluding most nonlinear static procedures. On the other hand, the presented methodology remains suitable for the majority of mid- and lowrise buildings since these configurations are characterized by a much simpler seismic behavior.

Despite the fact that, in principle, the analyst is allowed to choose any structural analysis process that is capable of assessing the structural response from elasticity to global collapse, for the case of highrise buildings, Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell, 2002) is proposed as the benchmark methodology. Furthermore, the important task of selecting a single Intensity Measure (IM) across the class, so as to parameterize the IDA results and consequently the vulnerability functions of the index buildings as well as the class vulnerability, will be addressed. In general, assuring efficiency and sufficiency for each building solely through the selected IM is more meaningful for a vulnerability assessment of a building class: Ground motion selection techniques are not fully applicable for the wide distribution of site hazard conditions encountered for each building.

Following the evaluation of the uncertain structural response in terms of story drifts and floor accelerations, the study proceeds to its main goal that is the vulnerability and loss assessment. This will be built upon the component-based FEMA P-58 approach (ATC, 2012) but simplified in such a manner so as to minimize the invested effort. Hence, contrary to FEMA P-58 that accounts for every single element in the buildings that is likely to be damaged, we will consider here only the top elements (structural, non-structural and contents), which are those contributing most to the construction cost of the index buildings. This simplification will allow us to perform a more rapid loss assessment. The end product will be a set of vulnerability curves for each one of the index buildings that provide the damage factor (i.e. top components repair cost divided by the total construction cost new of the building) along the selected common IM. By taking the weighted moments of the index building vulnerability functions (i.e. considering the weight of each building within the class) we will finally derive the uncertain vulnerability function of the class in a suitable form for application within the GEM framework.

CLASS DESCRIPTION AND SAMPLING

The first step to take in performing an analytical seismic vulnerability assessment of buildings, is the definition of the asset class of interest. This process essentially involves defining the asset class by means of several attributes, such as the height range, the structural material, the vertical and/or plan irregularity, the construction era, the lateral-load resisting system or any other feature that is considered to be important in structural response contribution terms. The class of interest for this case study is the set of reinforced-concrete moment-resisting frame (RCMRF) highrise buildings, built in the US in high-seismicity regions. The main features differentiating buildings within the class were considered to be:

a. The building height, defined as the number of stories.
b. The vertical irregularity ratio, defined as the ratio of the first story height to the (constant) height of the higher stories.
c. The plan irregularity ratio. This is defined for a roughly L-shaped plan, as the maximum of x_{max}/x_{min} and y_{max}/y_{min}, where x_{max} and y_{max} are the longer sides and x_{min}, y_{min} are the shorter sides of the L-shape in each respective horizontal axis.

d. The design base shear, as determined by the code-based value of spectral acceleration at 1.0sec, termed *SD*1 in US codes, that anchors the design spectrum for tall buildings.

There are three options for level of effort when choosing index buildings under the GEM analytical vulnerability methodology. The most computationally intensive approach is illustrated here. It begins by defining 5 or more index buildings (samples) of the asset class. By "index buildings" is meant specimens of the asset class that span the dimensions of building height, vertical irregularity, plan irregularity and design-strength such that the joint moments (mean, variance, etc.) of the weighted samples approximate the same moments of the distribution of the class as a whole.

To determine the set of buildings needed for characterizing the class, we need statistical data that provide a probabilistic description of the actual population. For their macroscopic characteristics (1-3) this is based on a sample of buildings catalogued by Cho and Porter (2014). For assessing the distribution of *SD*1 in high-seismicity zones, a comprehensive catalogue of US highrise buildings has been extracted from the Emporis highrise building database (www.emporis.com) and appropriately processed. To minimize the number of samples needed to represent the population of highrise RCMRFs, the set of representative index buildings was selected using moment-matching (Ching *et al*, 2009). This is an algorithmic procedure for determining a set of $2k+1$ samples that preserves the statistical characteristics of a population with k important features, where $k = 4$ in our case. One index building characterizes the center of the population while two more are needed to define the properties of each of the k features. The ensemble of these k additional pairs of index buildings, used to define the deviation from the central point, are termed the sigma set.

The moment-matching algorithm was run for each of the $k = 4$ features to register their relative importance. Plan irregularity proved to be of minor significance, contributing less than 1% to the total weight. Thus, only 3 features were used, for a total of 7 structures, numbered from 0 through 6 and shown in Table 1. The 1D moment-matching process for defining the three delta functions associated with each one of the key features, produces only minor error and thus the match between the first five moments (formally, the expected value of X, X^2,…X^5) and the original data is almost exact for each of the three dimensions considered. The accuracy of the proposed methodology mostly stems from the fact that the process does not assume any standard parameter probability distributions (e.g. uniform, normal, lognormal) for computing the five unknowns (i.e. three positions and two weights) associated with the three delta functions that describe each key feature of the index buildings. It should be noted, that initially moment-matching suggested the following three values for the number of stories in the index buildings: 7, 12 and 33. However, the 33-story index building was replaced by a 20-story building with the cost of just a small increase in the associated error. This constraint with respect to the building height was set to allow using a number of predesigned structures (Haselton *et al*, 2011) that can be easily modified to fit the needs of this study.

Table 1. Features *X*1, *X*2, *X*3 and moment matching weights for the seven index buildings

Index	No of stories, *X*1	Vertical irregularity, *X*2	Code design level, *X*3	Weight
No0	12	1.744	0.6g*	0.1475
No1	7	1.744	0.6g*	0.0962
No2	20	1.744	0.6g*	0.0787
No3	12	1.150	0.6g*	0.3152
No4	12	2.745	0.6g*	0.0425
No5	12	1.744	0.26g*	0.1700
No6	12	1.744	0.97g*	0.1500

* *SD*1 for site class D

MODELING

With respect to structural modeling it can be said that any detailing level that is able to capture the dominant failure modes of the index buildings is suitable for application within a vulnerability assessment study. The aforementioned statement however, disregards potential limitations in time and computational resources, especially when dealing with a set of buildings rather than a single one. In that case, the analyst faces the challenge of going against the stream and instead of adopting sophisticated structural models to espouse simpler ones, a decision that is further supported by the broad variability within the class that can render meaningless the more complex modeling approaches. In the past, several studies have demonstrated that, under certain circumstances (e.g. regular building configurations, first-mode dominated structures), simplified modeling can be used for fragility evaluation without significant loss of accuracy (e.g. Jeong and Elnashai, 2007; Kazantzi *et al*, 2011). For the Global Earthquake Model, three levels of model complexity are proposed, offering three distinct choices of structural detail:

a) Level A (MDOF): A detailed component-by-component 2D or 3D multi-degree-of-freedom (MDOF) model of a structure, including elements for each identified lateral-load resisting component in the building, e.g. columns, beams, infills, walls, etc.
b) Level B (Stick): A simplified story-by-story 2D stick representation of a building.
c) Level C (SDOF): A single-degree-of-freedom (SDOF) representation by a 1D linear or nonlinear spring together with a predefined displacement and acceleration profile shape.

Table 2. Basic properties of the seven RC moment-resisting index frames

Building ID	Nos0,5,6	No1	No2	No3	No4
No of stories, i	12	7	20	12	12
Height of first story, h_1 (m)	6.911	6.911	6.911	4.558	10.878
Height of 2nd and above stories, h_i (m)	3.962	3.962	3.962	3.962	3.962
$X = h_1/h_i$	1.744	1.744	1.744	1.150	2.745
Total height (m)	50.5	30.7	82.2	48.1	54.5
Gravity/Lateral tributary areas	0.17	0.17	0.17	0.17	0.17
Tributary mass in each frame per floor (tn)	313	313	313	313	313
Fundamental period, T_1 (sec)	2.14	1.61	2.85	2.02	2.42

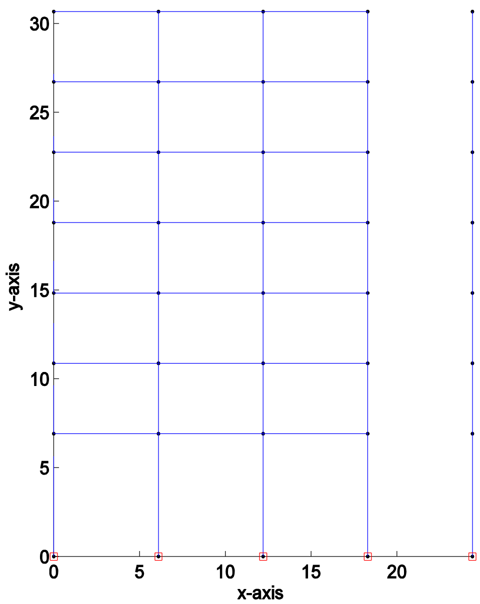


Figure 1. Model idealization of the No1, 7-story perimeter archetype RC frame. On the far right is the leaning column that is kinematically constrained at each floor with the three-bay frame on the left. Thus, all nodes at the same height have the same horizontal displacement at all times

For the case at hand, the simplest, yet efficient, representation of the index buildings was considered to be a 2D model idealization of the MDOF structure (i.e. Level A). The 2D modeling is justified by the insignificant plan asymmetry encountered by the index buildings. A Level B model is not recommended for slender highrise buildings (e.g. buildings with their height more than three times their width), since irregular shortening or lengthening of the columns may well violate the fundamental planar floor assumption and induce significant secondary moments (D'Ayala *et al*, 2013).

Level C is not suitable for highrise buildings for the apparent reason that their seismic performance in not anchored to a single mode of vibration.

Regarding the structural members, their behavior was depicted by lumped plasticity elements having their properties evaluated from empirical equations proposed by Panagiotakos and Fardis (2001). Adopting lumped plasticity elements, as opposed to the more sophisticated fiber section models (especially when close to collapse), was considered to be a reasonable compromise for a vulnerability assessment. Nevertheless, some adjustments were made with respect to the initial section stiffness in order to account for the concrete cracking that is not readily depicted otherwise. With respect to the nonlinear characteristics of the structural elements these were assumed to have an trilinear backbone with elastic, hardening and negative stiffness branches that ends at a predefined ultimate ductility level.

The seven index buildings were obtained on the basis of three perimeter frames that were selected and accordingly modified from a database of thirty (30) archetypes. This database was developed by Haselton *et al*. (2011) and was also used as the test bed in the example applications of FEMA P695 (ATC, 2009). To properly account for the destabilizing P-Delta effects, the perimeter frames were assumed to carry only a fraction of the tributary gravity loads that were considered for evaluating the seismic floor masses whereas, the remaining portion was applied on a leaning column. This is anticipated to be a reasonable assumption, which also partially accounts for the contribution of the internal gravity frames on the lateral resisting mechanism of the buildings: The double-curvature bending of columns is considered, while any framing action provided by the gravity frame beams is neglected. The main properties of the seven index buildings are summarized in Table 2 whereas Fig.1 depicts the model idealization of No1, the 7-story perimeter frame, along with its corresponding P-Δ leaning column.

IDA FUNDAMENTALS

For evaluating the seismic response, the Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell, 2002) is adopted. IDA is a powerful tool of structural analysis that involves performing a series of nonlinear time-history analyses for a suite of ground motion records scaled at increasing intensity levels. The intensity level range should be chosen to reveal the entire structural behavior, from yielding to global dynamic instability, i.e., collapse. To define the IDA curves, two scalars are needed, these being an intensity measure (IM) to represent the severity of the seismic input and an engineering demand parameter (EDP) to monitor the structural response. For the present study, a number of different IMs were used for illustrating their efficiency, whereas only two classes of EDPs are needed for loss assessment as per the GEM guidelines: The peak interstory drift ratio (IDR) at each story and the peak floor acceleration (PFA) at each floor. These are all that is needed from structural analysis for assessing structural, non-structural and content losses. Although computationally demanding, the IDA methodology remains, for the time being, among the best candidates for undertaking a thorough evaluation of the seismic structural performance for vulnerability assessment.

The ground motion records needed for the IDAs comprise the far-field record set of FEMA P695 (ATC, 2009) amounting to 22 records with two horizontal components (i.e. 44 individual components in total). The records were selected from the PEER NGA database (PEER, 2006) and were recorded either on stiff or very stiff soil sites. They only include ground motions which are recorded at least 10km far from the fault rupture and bear no marks of directivity. Furthermore, no more than two strongest records were considered from any one earthquake event to prevent event bias.

IM SELECTION

The selection of an appropriate intensity measure (IM) is an important task towards the development of analytical seismic vulnerability functions, either for a single building or for a set of index structures. The IM essentially governs the bias and variance inherent in evaluating the structural demand for given levels of intensity. Thus, as defined by Luco and Cornell (2007), the two most important properties of an IM are efficiency and sufficiency. Sufficient is an intensity measure that renders the structural response independent of any other seismological or ground motion characteristic, most importantly the earthquake magnitude, distance and the ground motion epsilon, ε. The epsilon of a record at a given period T measures the difference of the log of the spectral acceleration, $\log S_a(T)$, of a recorded ground motion from the mean $\log S_a(T)$ normalized by the standard deviation, the latter two predicted from a ground motion prediction equation (GMPE, e.g., Abrahamson and Silva, 1997).

An IM is deemed to be efficient when it is highly correlated to the structural response (Luco and Cornell, 2007), thus reducing its variability from record to record. In general, using a more efficient intensity measure gives a clear computational advantage, since with fewer records, and consequently fewer nonlinear dynamic analyses, we can achieve the same level of confidence with respect to the response estimates (Vamvatsikos and Cornell, 2005). The gain is even higher when assessing the seismic vulnerability for a building class, since the analysis process, which involves several nonlinear dynamic analyses, needs to be repeated for every index building within the class. It should be noted that despite this reduction in dispersion gained by a more efficient IM, this does not mean that the overall variability is reduced. Instead, it is simply shifted to a different level within the risk assessment, and in particular to the seismic hazard curve definition. A more efficient IM is invariably more structure specific and thus receives higher dispersions when trying to define an appropriate GMPE. In other words, no matter the adopted intensity measure, as long as sufficiency is maintained we should end up with the same overall variability at the end of the risk assessment, i.e., after convolving the vulnerability curve with the hazard.

While a user with abundant computational resources might be able to tolerate an inefficient IM, sufficiency still remains a *sine qua non* condition. An insufficient IM can result in an estimate of seismic performance or loss that depends on the ground motions that were selected for running the nonlinear dynamic analyses (Luco and Cornell, 2007). Thus, bias may enter the process, shifting the estimate of seismic risk away from its true distribution. Considering a set of structures, as opposed to a single building, increases the requirements placed on the IM. Now, the selected single IM should remain efficient and (more importantly) sufficient for the entire class, a prerequisite that is not easily met: A highly sufficient IM is invariably structure specific, yet it now needs to be tied to any number of index buildings, rather than just one.

$S_a(T_1)$ is often considered to be a relatively sufficient and efficient intensity measure (Shome *et al*, 1998), and certainly an improvement over the peak ground acceleration. Still, this is only accurate for low/mid-rise buildings and for limit-states that do not involve large ductilities (say higher than 3). The reason is that it is limited to the first-mode and hence does not account for the contribution of the higher modes or even the elongation of the first-mode after yielding (Vamvatsikos and Cornell, 2005). Nevertheless, even the simple $S_a(T_1)$ does not fit the description of a common IM for all buildings within the class, as it is too structure specific: Each building has its own first-mode period. A simple remedy is to choose a single common period T that can be considered representative of the class. Two potential candidates are $S_a(1\text{sec})$ and $S_a(T_{1m})$, where T_{1m} is the mean (or median) of the first-mode periods of all index buildings. For the case at hand, the mean period was found to be T_{1m} = 2.2sec. $S_a(1\text{sec})$ is an IM that has seen much use in existing vulnerability/fragility studies for highrise buildings but its efficiency is highly questionable. Still, it will be used just for comparison purposes.

On account of single buildings, Cordova *et al*. (2000) introduced the S_{agm} that was initially defined as the geometric mean of two spectral acceleration components evaluated at two period levels, these being the fundamental period (T_1) and a period that is two times the fundamental ($2T_1$). The latter period level was introduced so as to account for the period elongation associated with the structural damage. In the same ballpark, Vamvatsikos and Cornell (2005) also demonstrated that considering higher modes by using two or even three spectral values, results in more efficient intensity measures for the case of highrise buildings. In view of the aforementioned findings, the second class of intensity measures that was used in this study and actually recommended for the vulnerability assessment of highrise buildings, is the $S_{agm}(T_\text{i})$. The $S_{agm}(T_\text{i})$ is defined as the geometric mean of spectral accelerations values $S_a(T_\text{i})$ estimated at several periods T_i that may span the following ranges:

1. Five logarithmically spaced T_i periods over the $[T_{2\text{m}}, 1.5T_{1\text{m}}]$ range, where T_{2m} and T_{1m} are the mean T_2 and T_1 periods, respectively, of the index buildings, which in the case at hand result to: $(T_\text{i})_\text{I}$=[0.7 1.0 1.5 2.2 3.3],

2. Seven logarithmically spaced T_i periods over the [minT_2, 1.5maxT_1] range, which for the highrise set of buildings is equal to $(T_{\mathrm{i}})_2$=[0.7 0.8 1.0 1.2 1.5 1.8 2.2],
3. Five linearly spaced T_{i} periods over the [T_{2m}, 1.5T_{1m}] range, which for the highrise set of buildings is equal to $(T_{\mathrm{i}})_3$=[0.7 1.4 2.0 2.7 3.3],
4. Four T_{i} periods defined as [T_{2m}, min [(T_{2m}+T_{1m})/2, 1.5T_{2m}], $\mathrm{T}_{1\mathrm{m}}$, 1.5T_{1m}], which for the highrise set of buildings is equal to $(T_{\mathrm{i}})_4$=[0.7 1.0 2.2 3.3],
5. Five T_{i} periods defined as [T_{2m}, min [(T_{2m}+T_{1m})/2, 1.5T_{2m}], T_{1m}, 1.5T_{1m}, 2T_{1m}], which for the highrise set of buildings is equal to $(T_{\mathrm{i}})_5$=[0.7 1.0 2.2 3.3 4.4].

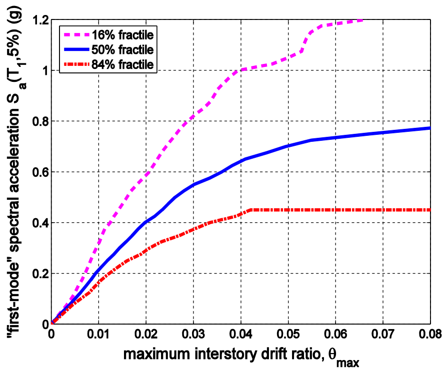
IDA ANALYSIS RESULTS

IDA was applied to each of the seven index buildings for the 22x2 ground motion accelerograms using the hunt&fill algorithm to achieve a consistent number of 12 nonlinear dynamic analyses per record. In each case, the analysis was run up to global dynamic instability. For each run the peak floor acceleration and interstory drift values at each floor or story were collected and then interpolated to generate the required "stripes" of each EDP at given levels of the IM. To gain some insight into the behavior of the structural models, Fig.2 presents an example of the results, in the form of the 16,50,84% fractile IDA curves for the maximum interstory drift, i.e. the maximum of all peak interstory drifts recorder at any story. These are defined at each level of IM by taking the 16,50,84% fractiles from the 44 recorded EDP values, one per each ground motion record. It should be noted that the results presented in Fig.2 are not directly comparable due to the use of a different $S_a(T_1)$ for each building (at least wherever the first-mode period has changed). For this reason, we shall transform our results to a common IM that can be used for defining the vulnerability function of the class.

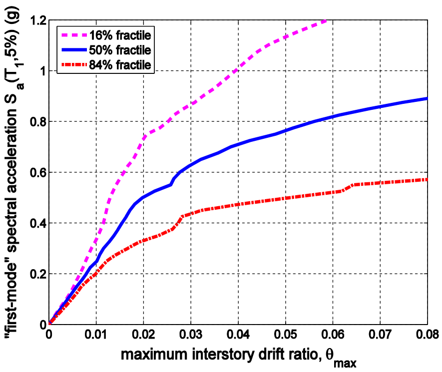
COMPARISON OF DIFFERENT IMs

The testing of candidate IMs for efficiency and sufficiency can be performed *a posteriori* and for any number of IMs without incurring any computational cost: The same IDA results are simply reused and replotted. It is thus suggested that an informed analyst may undertake such a study if he/she suspects that a certain IM might be a better choice for a given class of structures than the one suggested. The proposed methodology differs from similar studies that have appeared in the literature, e.g., Baker and Cornell (2005), Tothong and Luco (2007), Bianchini *et al.* (2009), in two important aspects, namely (a) using an IM given EDP (IM|EDP) basis and (b) employing all IDR and PFA values at each story. Working on an IM|EDP basis essentially translates to using "vertical stripes" of points in Fig.2, produced as cross-sections of the 44 IDA curves with a vertical line signifying a given EDP value. Thus, for each EDP type and for any number of its values, one may estimate IM "capacity values" required by each corresponding record to reach these targets. This has the obvious advantage of allowing a detailed view of efficiency/sufficiency that can reach all the way up to collapse (Vamvatsikos and Cornell, 2005). On the contrary, past studies have often relied on processing directly EDP values (typically for given levels of the IM), thus being forced to stay away from global collapse where response becomes essentially infinite or undefined.

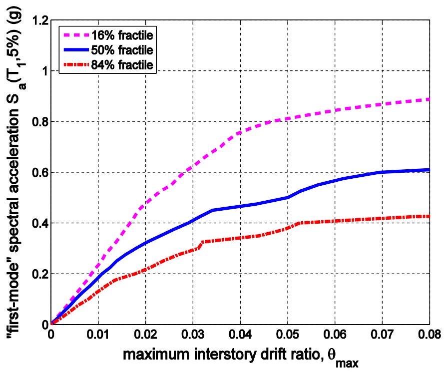
Efficiency is tested by estimating the dispersion β_{IM} of the IM|EDP values, i.e. the standard deviation of the log of the IM capacities for a range of EDP values. Lower dispersions mean higher efficiency. The ensemble results are shown for the No0 12-story in Fig.3, separately for acceleration and drift EDPs. To summarize the results, for each value of acceleration or drift we plot the maximum over all stories (including the roof and overall max drift where applicable) of the dispersion (Fig.3a and Fig.3c). This signifies the worst performance along the height of the building. The average β_{IM} along the height of the building is also plotted (Fig.3b and Fig.3d) to indicate the expected performance of each IM at each level of deformation. Despite showing the results from a single building only, the same trends persist in each of the seven index buildings, therefore the discussion to follow should be considered to be applicable to each and every one.



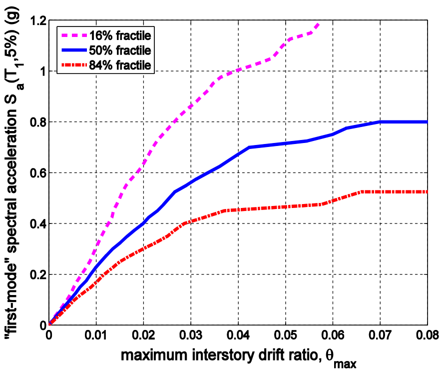
(a) No0 12-story



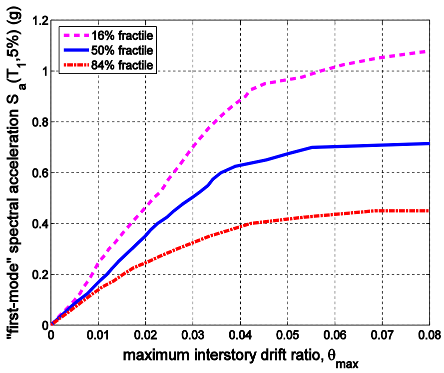
(b) No1 7-story



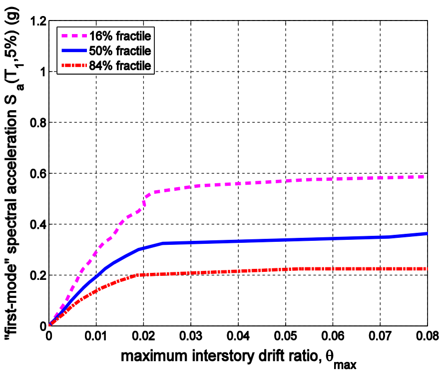
(c) No2 20-story



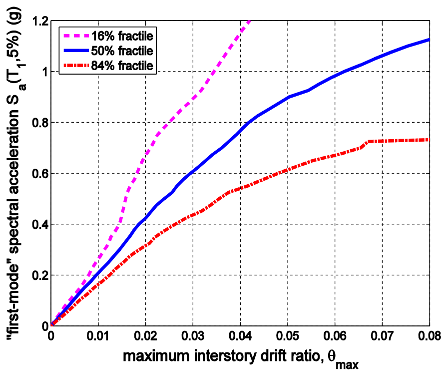
(d) No3 12-story (No0 short)



(e) No4 12-story (No0 tall)

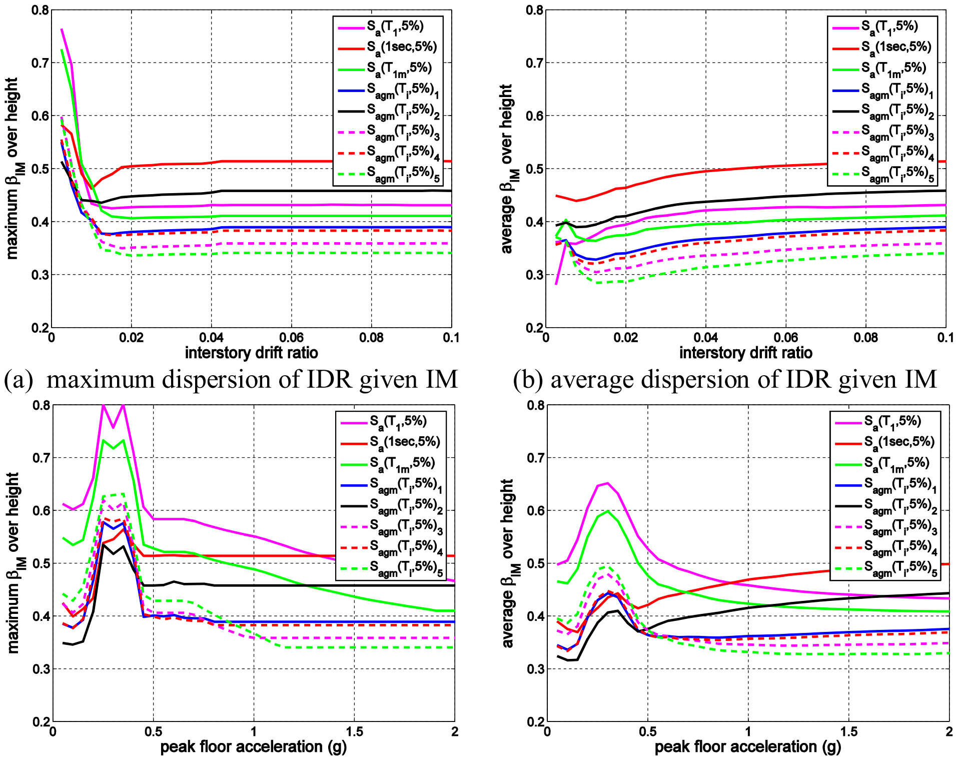


(f) No5 12-story (No0 $SD1_{low}$)



(g) No6 12-story (No0 $SD1_{high}$)

Figure 2. Summarization of the IDA curves into 16,50,84% fractile curves of the maximum interstory drift



(a) maximum dispersion of IDR given IM (b) average dispersion of IDR given IM

(c) maximum dispersion of PFA given IM (d) average dispersion of PFA given IM

Figure 3. Maximum and average dispersions of the IM for given values of the IDR and PFA response of the No0 12-story index building considering eight intensity measures. The averages and the maxima are calculated by considering the corresponding peak values at each story or level, together with the overall maximum IDR over all stories and the roof drift where applicable

Evidently, among the least efficient IMs across the examined limit state range, is the S_a(1sec,5%), especially when considering IDR as a response measure. The dispersions achieved by $S_a(T_1)$, which is useful for single buildings only due to changes in T_1, and $S_a(T_{1\mathrm{m}})$, which can be used for a set of index buildings, were also proven to be high, rendering their use relatively expensive: More ground motion records will need to be employed for a good estimate of the distribution of vulnerability. Among the remaining IMs and considering their performance both in elastic and inelastic regions for IDR and PFA, the $S_{agm}(T_{\mathrm{i}})_4$, was proven to provide relatively stable dispersion estimates. It employs four periods, the $T_{1\mathrm{m}}$ and the $T_{2\mathrm{m}}$ and their "elongated" versions by a factor of 50%, thus equally favoring the first and second mode. $S_{agm}(T_{\mathrm{i}})_5$, having one more longer-than-T_1 period is an equally good choice, showing better efficiency practically everywhere with the exception of the PFA hump where it performs slightly worse than other IMs. Similarly good results have been derived for these two composite IMs with respect to sufficiency as detailed in Kazantzi *et al*. (2014).

In conclusion, it seems that a successful IM is created by specifying an appropriate period range that includes periods both above and below T_{1m} and uses the geometric mean of the spectral accelerations evaluated at these periods (see also Bianchini *et al*, 2009; Tsantaki and Adam, 2013). Nevertheless, by all means, the selection of an efficient and sufficient IM remains an open research area where the last word has not yet been said; future research is expected to offer further insights into what constitutes a good IM.

VULNERABILITY ESTIMATION

When estimating seismic losses, in order to inject the needed variability, one also defines three variants of each index building: one variant with relatively rugged components, one with typical components, one with relatively fragile components. Only the top 6 or so nonstructural/content component types and the top 1 or 2 structural component types are considered. By "top components" is meant the components that contribute most to construction cost new. Component types are categorized based on the FEMA P-58 taxonomy (ATC, 2012), which accounts for installation conditions and other details that strongly affect component fragility or repair cost. The selection of detailed components requires experience in quantity takeoff (from construction contracting) and familiarity with seismic installation conditions; consult a contractor or a cost manual such as RSMeans (2012) if needed. In any case, each of the 7 index buildings is analyzed with poor, typical, and superior-quality variants, for a total of 21 variants.

The values of peak floor accelerations at each floor or roof diaphragm and peak transient drift ratios at each story, captured via IDA, are input to fragility functions for each component at each floor (for acceleration-sensitive components) or story (for drift-sensitive components). One uses Monte Carlo methods to simulate ground motion time history, damage for each component, and repair costs per damaged component type and damage state. Total damage factor (*DF*, repair cost as a fraction of replacement cost new) in any simulation is given by the following equation, in which V denotes the replacement cost new of the building, f denotes the fraction of V represented by the component types in the inventory, a is an index to floor level, N_a is the number of diaphragms, c is an index to component types, N_c is the number of component types considered, d is an index to damage states for a given component type, N_d is the number of possible damage states, $n(a,c,d)$ is the number of damaged components at floor a, type c, in damage state d, and $u(c,d)$ is the unit cost to repair a component of type c from damage state d.

$$DF = \frac{1}{V \cdot f} \sum_{a}^{N_a} \sum_{c}^{N_c} \sum_{d}^{N_d} n\left(a,c,d\right) \cdot u\left(c,d\right) \tag{1}$$

One calculates DF for each of many simulations for each combination of structural model and component set at each level of ground motion intensity, and captures mean damage factor (MDF) and coefficient of variation (COV) as a function of ground motion intensity. The example that follows used 1,000 simulations for each index building and variant at each level of ground motion intensity. One equally weights the poor, typical, and superior-quality variants to estimate the MDF and COV for each index building and applies the moment-matching weights to calculate the MDF and COV for the class as a whole. The vulnerability functions shown in Fig.4 already aggregate the quality-level variants of each index building. Note that the dashed line denotes the coefficient of variation of damage factor for the asset class, accounting for all of the variabilities discussed here. The curves are wiggly, partly because of the limited sample of 44 ground motion time histories in the IDA and partly because we limited the Monte Carlo simulation to 1,000 iterations; we suspect more because of the former than the latter limitation. If this is an issue, smooth the curves can be considered.

Fig.4 suggests several observations; two of the most noteworthy are as follows. First, even within the fairly narrowly defined asset class, there is a great deal of variability of vulnerability, with the most vulnerability index building being three to four times as vulnerable as the least, even after averaging over three variants. The coefficient of variation exceeds 1.0 at ground motion as high as Sa(1.0 sec) $\leq$ 0.6g, a range that represents most actual damage earthquake cases. Experts tend to underestimate variability (e.g., Tversky and Kahneman, 1974) so the COV shown here is far greater than that implied by, say, ATC-13 (ATC, 1985). Second, all three attributes that vary between index building matter: number of stories, design base shear, and to a lesser extent degree of vertical irregularity. Not shown is how much component quality matters: a lot. The least and most vulnerable variants among all 21 specimens can differ in mean damage factor by two orders of magnitude, a fact that should not surprise engineers who have performed post-earthquake surveys and observed heavily damaged buildings near relatively undamaged ones of the same general category.

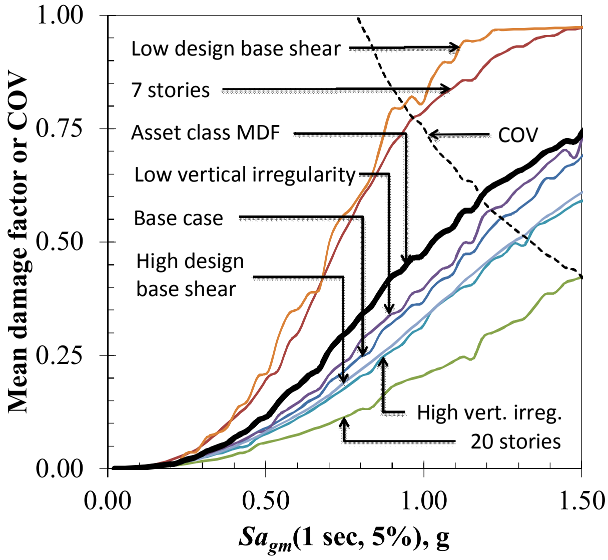


Figure 4. Mean vulnerability functions for 7 index buildings and mean and coefficient of variation of vulnerability for the class

The analysis presented here was performed in approximately 34 to 56 person-hours. This includes 18-24 person-hours for setting up and executing the structural analysis and 16-32 for the cost estimation, but it does not incorporate the construction of the structural models, which was probably the most time-consuming aspect of the analysis, and the construction of a spreadsheet to perform the Monte Carlo simulation. In short, it is practical.

CONCLUSIONS

A practical methodology has been presented for performing analytical vulnerability assessment for highrise building classes. Significant novel features of the proposed approach include:

a) Using moment-matching to select representative index buildings,
b) The use of simple structural models together with IDA for performing structural assessment,
c) The introduction of the geometric mean of spectral accelerations at adjacent periods as a sufficient and efficient intensity measure across an entire building class,
d) The use of a reduced list of “top components” that need to be taken into account for assessing the damage factor,
e) Monte Carlo simulation to propagate the uncertainty from different realizations of each index building to the class vulnerability results.

As an example, it has been applied to modern office higrise reinforced-concrete moment-resisting frames built in high-seismicity regions of USA, achieving good results within a reasonable time-frame.

ACKNOWLEDGEMENT

This work has been supported by the GEM Foundation through the GEM Vulnerability Consortium and co-financed by Greece and the European Union (European Social Fund) through the Operational Program “Human Resources Development” of the National Strategic Framework (NSRF) 2007-2013.

REFERENCES

Abrahamson NA, Silva WJ (1997) “Empirical response spectral attenuation relations for shallow crustal earthquakes”, *Seismological Research Letters*, 68(1): 94–127.

Applied Technology Council (1985) ATC-13, Earthquake Damage Evaluation Data for California, Redwood City, CA.
Applied Technology Council (2009) FEMA P695, Quantification of building seismic performance factors, Federal Emergency Management Agency, Washington, DC.
Applied Technology Council (2012) FEMA P-58-1, Seismic Performance Assessment of Buildings Volume 1 – Methodology, Federal Emergency Management Agency, Washington DC.
Baker JW, Cornell CA (2005) "A vector-valued ground motion intensity measure consisting of spectral acceleration and epsilon", *Earthquake Engineering and Structural Dynamics*, 34(10): 1193–1217.
Bianchini M, Diotallevi PP, Baker JW (2009) "Predictions of inelastic structural response using an average of spectral accelerations", *Proceedings of the 10th International Conference on Structural Safety & Reliability (ICOSSAR)*, Osaka, Japan.
Ching JY, Porter KA, Beck JL (2009) "Propagating uncertainties for loss estimation in performance-based earthquake engineering using moment matching", *Structure and Infrastructure Engineering*, 5(3):245–262.
Cho IH, Porter KA (2014). "Modeling building classes using moment matching", *Earthquake Spectra*, accepted.
Cordova PP, Deierlein GG, Mehanny SS, Cornell CA (2000) "Development of a two-parameter seismic intensity measure and probabilistic assessment procedure", *Proceedings of the 2nd US-Japan Workshop on Performance-based Earthquake Engineering Methodology for RC Building Structures*, Sapporo, Hokkaido.
D'Ayala D, Meslem A, Vamvatsikos D, Porter K, Rossetto T, Crowley H, Silva V (2013) Guidelines for analytical vulnerability assessment, Global Vulnerability Consortium, Available from: www.nexus.globalquakemodel.org/gem-vulnerability/posts/.
Haselton CB, Liel AB, Deierlein GG, Dean BS, Chou JH (2011) "Seismic collapse safety of reinforced concrete buildings. I: Assessment of ductile moment frames", *Journal of Structural Engineering*, 137(4):481–491; (http://myweb.csuchico.edu/~chaselton/research/research_databases/sd_db.php).
Jeong SH, Elnashai AS (2007) "Probabilistic fragility analysis parameterized by fundamental response quantities", *Engineering Structures*, 29(6):1238–1251.
Kazantzi AK, Righiniotis TD, Chryssanthopoulos MK (2011) "A simplified fragility methodology for regular steel MRFs", *Journal of Earthquake Engineering*, 15(3): 390–403.
Kazantzi AK, Vamvatsikos D, Porter K, Cho I (2014) Developing analytical vulnerability curves for Western US highrise RC frame buildings, Global Vulnerability Consortium, www.nexus.globalquakemodel.org/gem-vulnerability/posts/.
Luco N, Cornell CA (2007) "Structure-specific scalar intensity measures for near-source and ordinary earthquake ground motions", *Earthquake Spectra*, 23(2): 357–392.
Panagiotakos TB, Fardis MN (2001) "Deformations of reinforced concrete members at yielding and ultimate", *ACI Structural Journal*, 98(2): 135–148.
PEER (2006) PEER NGA Database, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, http://peer.berkeley.edu/nga/.
Porter K, Cho I (2013) "Characterizing a building class via key features and index buildings for class-level vulnerability functions", *Proceedings of the 11th International Conference on Structural Safety & Reliability (ICOSSAR)*, Columbia University, New York, NY.
Porter K, Farokhnia K, Vamvatsikos D, Cho I (2014) Analytical Derivation of Seismic Vulnerability Functions for Highrise Buildings, Global Vulnerability Consortium, www.nexus.globalquakemodel.org/gem-vulnerability/posts/.
Reitherman R, Cobeen K (2003) Design documentation of woodframe project index buildings, CUREE Publication W-29, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.
RSMeans Co., Inc. (2012) Square Foot Costs 2012, 33rd Edition, Kingston, MA.
Shome N, Cornell CA, Bazzurro P, Carballo JE (1998) "Earthquakes, records and nonlinear responses", *Earthquake Spectra*, 14(3): 469–500.
Tothong P, Luco N (2007) "Probabilistic seismic demand analysis using advanced ground motion intensity measures", *Earthquake Engineering and Structural Dynamics*, 36(13): 1837–1860.
Tsantaki S, Adam C (2013) "Collapse capacity spectra based on an improved intensity measure", *Proceedings of the 4th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2013)*, Kos, Greece.
Tversky A, Kahneman D (1974) "Judgment under uncertainty: heuristics and biases", *Science*, 185:1124-1131.
Vamvatsikos D, Cornell CA (2002) "Incremental dynamic analysis", *Earthquake Engineering and Structural Dynamics*, 31(3):491–514.
Vamvatsikos D, Cornell CA (2005) "Developing efficient scalar and vector intensity measures for IDA capacity estimation by incorporating elastic spectral shape information", *Earthquake Engineering and Structural Dynamics*, 34(13): 1573–1600.