Tall Building Analytical Seismic Vulnerability Functions for the Global Earthquake Model

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ABSTRACT: Guidelines for developing analytical seismic vulnerability functions are proposed for use within the framework of the Global Earthquake Model (GEM). Emphasis is on high-rise buildings and cases where the analyst has the skills and time to perform nonlinear dynamic analysis. The basis for this effort is formed by the key components of the state-of-art PEER/ATC-58 methodology for loss assessment. Simplified modeling options are proposed to reduce model complexity and ease application for building ensembles. Nonlinear dynamic analysis is employed for the estimation of local story drift and absolute acceleration response to evaluate both structural and non-structural losses. Important sources of uncertainty are identified and propagated using simplified methods based on moment-matching to reduce the computational load. The resulting guidelines allow the generation of vulnerability functions for a class of high-rise buildings with a reasonable amount of effort by an informed engineer.

1 INTRODUCTION

The Global Earthquake Model (GEM) is a grand effort to offer a comprehensive open-source tool for loss assessment that can cover the entire globe. A vital component of this effort is the definition of physical vulnerability for different classes of buildings (Figure 1), a task that has been undertaken by a consortium of researchers from Europe, USA, and Australia. Separate thrusts of the project are geared towards defining building vulnerability functions based on (a) empirical data (b) expert opinion and (c) structural analysis. Our focus will be on the latter part with emphasis on high-rise structures and specifically on the development of guideline documents for estimating analytical vulnerability functions for assessing structural, non-structural and contents' losses.

The analytical vulnerability guidelines are developed as a hierarchy of approaches that can accommodate different levels of expertise. Our proposal thus accommodates several paths towards reasonable approximations that strike different compromises between the time committed and the accuracy achieved. The present work deals with the case where the analyst has the skills and time to perform nonlinear dynamic analysis. The minimum target is for a structural engineer with Master's level training and the ability to create simplified nonlinear structural models to be able to determine the vulnerability functions pertaining to structural response, damage or loss with reasonable effort. This has been defined as 20-40 man-hours for any single structure, and 80-160 man-hours for a class of high-rise buildings.

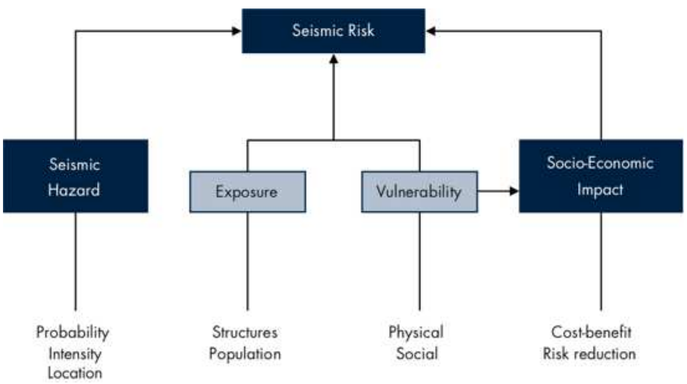

Figure 1. Conceptual framework for seismic risk assessment adopted by GEM (source: http://en.wikipedia.org/wiki/Global_Earthquake_Model).

2 FRAGILITY AND VULNERABILITY

First of all, let us clarify and differentiate the somewhat abstract terms of seismic fragility and vulnerability as they will be used henceforth. For our purposes, they are both functions whose only argument is a single scalar value of structural response or instrumental seismic intensity. The structural response variable of interest will be generally referred to as the engineering demand parameter (EDP), and the seismic intensity will be described by the intensity measure (IM), following the terminology adopted by the Pacific Earthquake Engineering Research (PEER) Center (Cornell & Krawinkler, 2000). Typical choices for the EDP will be the peak interstory drift (IDR) of any given story for structural and non-structural drift-sensitive damage, and the peak floor acceleration (PFA) for non-structural and contents' acceleration-sensitive damage. Suitable IM's are the 5%-damped first-mode (pseudo) spectral acceleration at the structure's first-mode period $S_a(T_1)$ or a geometric mean of spectral accelerations at n periods of interest,

$$S_{agm} = \left[\prod_{i=1}^{n} S_a(T_i) \right]^{1/n} . \tag{1}$$

Where fragility and vulnerability mainly differ, in our way of thinking, is at their output value.

Fragility is defined as a probability-valued function of IM or EDP. We use both component fragility functions and building fragility functions. Formally, component fragility will be defined as the cumulative distribution function of the engineering demand parameter (EDP) capacity of a building component (whether structural, nonstructural or content) to resist a particular damage state. The definitions of component damage states vary by component, but they are generally defined in terms of fairly objective observable physical damage associated with a particular repair effort. Essentially, this is a function that outputs the probability of violating a component's damage state for input values of the EDP. Story or building fragility is defined in exactly the same way, only now using IM as the argument and referring to the damage state of an entire story or building. Two sample building damage states are collapse and red-tagging (damage to the degree that a post-earthquake building safety inspector would deem the building unsafe to enter or occupy).

There is a long tradition of developing fragility functions that have found excellent usage from the early days of risk assessment in the nuclear industry (Kennedy & Ravindra 1984) up to more recent applications for the seismic assessment of buildings (e.g., Porter et al., 2007, Jeong & Elnashai 2007, Kazantzi et al, 2008, 2001). In some cases, fragility is geared towards defining a damage state for an entire building. This may still serve a useful role for some applications, for example tagging, but it tends to remove the necessary detail needed for loss estimation. Therein, story-level or even component-level damage states (and corresponding fragility functions) provide the fine-grained view of damage needed to reliably and transparently calculate losses and provide engineering insight into those losses. In addition, EDP capacities (or thresholds) for assigning damage states are notoriously difficult to determine for an entire building, let alone a building class. They are more naturally defined at the level of a component, where experimental results are available. Thus, building fragility is only a by-product of our methodology. Story- and component-level fragilities are more directly used, and even then only as a step on the road to estimating vulnerability functions.

Vulnerability is a loss-valued function of IM for an entire story or structure. With loss being an inherently probabilistic quantity, it is actually more accurate to discuss vulnerability functions for certain statistics of the loss in the story or the structure. Therefore, we typically operate on the basis of the mean vulnerability function and its dispersion given IM, or alternatively, the 16, 50 and 84% fractile values of vulnerability given the IM level. Henceforth, our focus will be on methods to deliver vulnerability functions for the central value (mean or median) and the dispersion of loss for a class of high-rise structures.

Estimating seismic vulnerability (or loss) functions has received a lot of attention in the last 40 years (e.g., Whitman 1973, Applied Technology Council 1985, Kircher & Whitman 1997) both for single and classes of buildings. One of the earliest attempts to use structural analysis and other engineering principles to estimate the relationship between ground motion and loss is Czarnecki's (1973) component-based approach. This was later enhanced with component test data by Kustu et al. (1982), and with nonlinear pseudostatic structural analysis and applied to the general population of US buildings by Kircher & Whitman (1997).

One of us (Porter et al. 2001) offered an early, fairly comprehensive analytical method that employs nonlinear dynamic analysis many times at each of many stripes of IM, estimates damage at the component level using component fragility functions, thoroughly quantifies and propagates uncertainties, and estimates uncertain building repair cost. The method, then called assembly-based vulnerability (ABV), applied to a single structure. ABV was further developed by a number of researchers formulating the PEER methodology, which we have come to call 2^{nd}-generation performance-based earthquake engineering (PBEE-2). Innovations of PEER PBEE-2 included: a fairly exhaustive taxonomy of building components detailed enough to distinguish important design and installation features; consideration of various competing intensity measures; methods for selecting and scaling ground-motion time histories; new tools for modelling structural response; component tests for damageability; and various insights facilitated by this new methodology.

For example, using PBEE-2 methods, Porter et al. (2004) found that component damageability and ground-motion time histories can have relatively strong influence on loss uncertainty while material properties and other uncertainties in the structural model may have relatively little. Aslani and Miranda

(2005) observed how collapse can contribute to probabilistic loss.

At present, the state-of-art in loss estimation *for a single structure* is represented by the ATC-58 (ATC 2012) methodology that implemented PEER PBEE-2 and added simulation software called PACT that includes a database of component fragility functions and repair cost estimates. For practical reasons, ATC-58 also simplified PEER PBEE-2 by eliminating all EDPs other than story drifts and floor accelerations. In PEER's PBEE-2 methodology, component fragility functions for structural damage could use a function of member force, deformation, or hysteretic energy dissipation as its argument.

Our approach will essentially parallel the ATC-58 guidelines, extending its validity to a *class of buildings* while at the same time incorporating simplified approaches and shortcuts wherever the computations become cumbersome.

3 INDEX BUILDINGS

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. Even when confined to high-rise buildings, there is ample space for variability within these boundaries. Selecting an appropriate sample set of buildings, referred to here as *index buildings* after the style of Reitherman and Cobeen (2003), to represent the class becomes central to the estimation of vulnerability. The amount of effort is proportional to the size of the sample, while the quality (i.e., accuracy) of the final results in both the central value and the dispersion depends on using as many as practical as efficiently as possible.

Having a strict target of up to 160 man-hours for the entire class, large sample sizes become unrealistic. Hence, three distinct options and corresponding levels of accuracy are offered that are later on tied to custom approaches for propagating uncertainty from the sample to the vulnerability. In short, the user is directed to choose between 1, 3 or 5 – 7 index buildings in total, and to choose among a range of uncertainty propagation procedures. Given a structural material, lateral force resisting system, height range, and era of construction, in order of relative importance, these index buildings should be selected according to (a) design base shear, (b) height (or number of stories), (c) degree of vertical irregularity and (d) degree of horizontal irregularity. Good knowledge of the statistics and dominant characteristics of the targeted building population are considered essential in selecting the properties of a representative sample. To acquire this knowledge can require a survey of buildings like the class of interest; see for example Porter and Cho (2013) for a survey of California concrete buildings.

Sample selection (the design of particular index buildings to represent the class) may be performed according to two distinct methods that directly correspond to the way that the individual results will be combined in the end. The first is a so-called star-design pattern that is meant to capture $k = 2$ or 3 significant class properties. It employs a central index building plus $2k$ (i.e., 4 or 6) variations representing high/low values in each dimension, or characteristic of interest. An example for a class with two significant properties ($k = 2$) appears in Figure 2. This is the suggested methodology for relatively homogeneous populations of buildings whose properties could be reasonably approximated by a unimodal joint probability distribution function (PDF) where the properties are relatively independent. Estimation of the overall probabilistic properties of the ensemble is then performed via the moment matching procedure, described in Porter and Cho (2013).

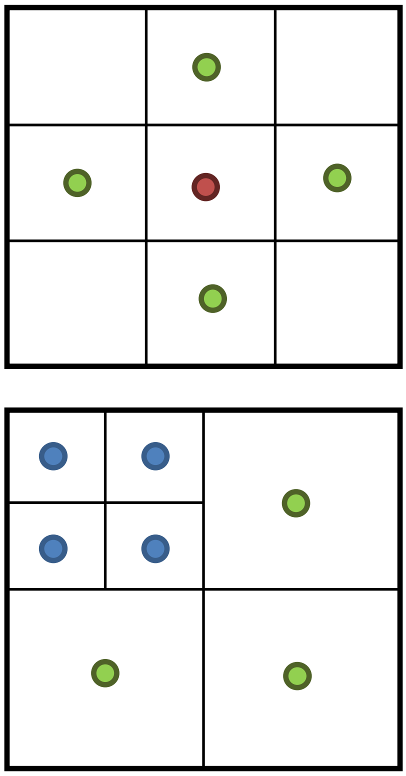

Figure 2. Conceptual representation of sampling via star design (top) versus subclass partitioning (bottom) for a class with two significant properties. The economical incomplete partitioning of star design is apparent.

The second method, the so-called subclass partitioning, involves partitioning the population into a set of collectively exhaustive and mutually exclusive subclasses of buildings. Each of the subclasses is represented by a single index building. This is recommended for populations that have strongly correlated properties or may be inhomogeneous in terms of their significant properties. This is the case for example when the corresponding distributions may be strongly bimodal or discontinuous.

In all cases the overall properties of the building population are approximated through the joint probability mass function (PMF) established by the index buildings. In simpler terms, this means that the median and standard deviation of the population properties are assessed by condensing the population to just the index buildings used, to each of which a certain weight is assigned. For subclass partitioning, this is estimated according to its actual membership (percentage of buildings it represents) in the entire population. When using moment-matching, the algorithm automatically assigns the weight to best represent the population distribution properties. Hence, the membership of each index building in the entire population can vary. This essentially opens up the option for fine-tuning the vulnerability function if appropriate exposure data is available for a specific region, as long as the individual index building vulnerability functions are available to the end user.

4 MODELING

The development of structural models for each index building is an important issue when deriving analytical vulnerability functions. The complexity of detailed modeling offers undeniable accuracy, yet it will often absorb most of the effort due to the multiple representative buildings employed. For high-rise buildings two levels of model complexity are suggested, offering two distinct choices of structural detail.

The first level entails a detailed 2D or 3D multi-degree-of-freedom model of each index building. The choice between 2D and 3D depends on the plan asymmetry characteristics of the building, the latter being most appropriate wherever significant eccentricity (torsion effects) are expected. Appropriate representation of the nonlinear behavior of all identified lateral-load resisting component in the building (columns, beams, walls, braces etc.) is essential. Material nonlinearity needs to be included at least at the level of a capped elastic-plastic constitutive relationship. This means at a minimum an elastic perfectly-plastic relationship of force-deformation, moment-rotation or stress-strain that contains a hard-coded ultimate ductility to simulate component failure (Figure 3). Appropriate representation is also need for any significant global or local geometric nonlinearities (e.g., P-Δ effects, brace buckling). In short, this is a modeling level that is roughly equivalent to the detail needed for assessing an individual building according to current seismic guidelines.

In general, only engineers well-versed in nonlinear structural modeling should follow this approach for vulnerability analysis; it is bound to be quite time-consuming for the average user. Furthermore, despite the inherent accuracy and reliability of such detailed models, their use in loss estimation for an entire class may not be always practical. The broad variability within the class means that individual details that have been painstakingly modeled will eventually disappear and only some macro-characteristics may dominate.

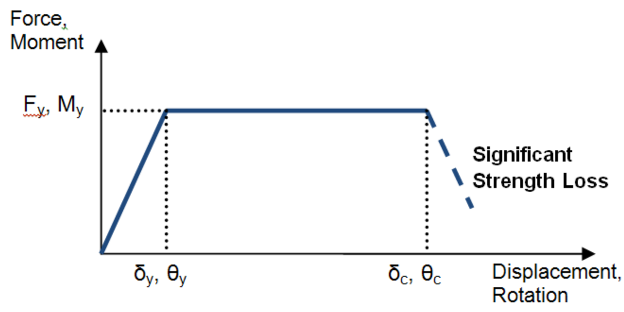

Figure 3. The capped elastic-plastic backbone is the simplest recommended force-deformation or moment-rotation backbone relationship for elements.

Thus, the simplicity of a 2D stick representation of a building becomes a cost-effective alternative. Our proposal is based on the concept of “fishbone” models pioneered by Luco et al. (2003) to represent moment-resisting frame buildings. This reduces a frame to a single column-line, each story rotationally restricted by two half-beams that are roller-supported at their opposing ends. This idea has been further simplified and generalized to represent both flexural and shear buildings as shown in Figure 4. It retains the column-line, comprising *N* columns and *N* nodes for *N* stories, each with 3 degrees of freedom (horizontal, vertical, rotational) in 2D space. The nodes are further restrained by *N* rotational springs representing the strength and stiffness of beams at each floor. All elements are nonlinear, at the very minimum having a capped elastoplastic behavior. Element characteristics can be easily derived using the aggregate stiffness of the columns, piers, walls or beams in each story together with the corresponding yield and ultimate displacements or rotations. Only translational story masses need be assigned to each node, while global P-Δ effects are explicitly taken into account.

By thus condensing the characteristics of each story into one column and one rotational spring, the stick model achieves excellent economy. While it can capture many of the salient features of modern buildings, especially height, vertical irregularities and flexural versus shear behavior, it cannot take into account any effects related to the two neglected horizontal dimensions. For example, the effect of column compression/tension due to the overturning moment, or any shear lag effects within a single beam-line are not captured. In addition, as with any 2D structure, 3D interaction effects are not modeled. This is of little importance for plan-symmetric structures with distinct lateral load-resisting systems in the two horizontal directions. It becomes increasingly important for plan-asymmetric structures or wherever the appearance of mass/shear center eccentricity causes torsion. It may also become an issue if the system strength in the two horizontal directions is strongly interacting, thus making biaxial shaking significantly more detrimental than uniaxial.

Such 3D effects can only be taken into account approximately by using theoretical or regression expressions to relate, e.g., the index of plan asymmetry to a reduction in the column and beam ductility capacities incorporated in the model. This is considered a far superior approach than using a direct "damage modifier", where the modification is applied on the EDP displacement (or acceleration) response of the model rather than its properties, leading to considerable difficulties in properly defining collapse. In other words, it is not easy to make such modifications influence the IM-level of collapse when applying them only in post-processing. By including them in the model properties, though, their integration becomes more natural. For example, it can be shown that a normalized plan eccentricity of e in one direction of a square-plan multi-bay frame leads to an increase of elastic base shear in the perpendicular direction by a factor of $b = (1+3e)$. Reducing the yield and ultimate ductility of both the beam spring and the column element in the corresponding eccentric stories yields a simple method to roughly account for this effect.

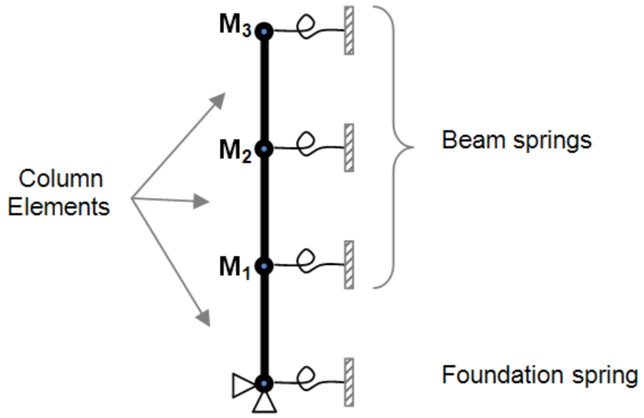

Figure 4. A three-story stick model, showing rotational beam-springs, column elements and floor masses M_1 – M_3.

5 STRUCTURAL ANALYSIS

The mainstay of current practical guidelines for seismic assessment is the nonlinear static procedure, based on the use of static pushover analysis. Unfortunately, this approach hinges on the approximation of a structure by its first mode, thus being unsuitable for high-rise structures. This is why only two approaches are considered for our purposes, namely, (a) a bare-bones approximation using a first-mode response spectrum concept with a generous margin of error and (b) nonlinear dynamic analysis. The latter is actually the only option that can be endorsed for general application and it will be our focus for the rest of this work. It necessitates some minimum of computational power, yet its application is relatively straightforward pending the resolution of two important issues: What IM to use and which ground motions to apply.

5.1 *Intensity measure selection*

It is important to note that regardless of the IM that may be used in the analysis of each individual index building, the results should always be mapped to a single scalar IM that is considered representative of the entire class. PGA may have been a popular option in past studies, but it is not considered viable even for a single high-rise index building in light of modern requirements of efficiency and sufficiency. Simply put, PGA is only adequate for the shortest of periods and fails to express the intensity of longer periods of motion that impact tall structures. $S_a(T, 5\%)$ instead is a much improved candidate, as it captures at least the fundamental period response. Yet it cannot account for the higher-modes that influence tall buildings, nor can it represent a broader period range that is characteristic of an entire class. Some "average T" would have to be used with mixed or unknown results.

An alternative route has been suggested for single buildings by Cordova et al. (2000), who introduced S_{agm}; at the time this was the geometric mean of only two spectral acceleration components at T_1 and $2T_1$, the latter added to account for period lengthening due to damage. This was an improvement over S_a for mid-rise structures. Vamvatsikos & Cornell (2005) also showed that including one or two higher modes also imparts a distinct advantage in terms of efficiency and sufficiency for high-rise structures. Finally, Bianchini et al. (2009) presented the benefits of using S_{agm} to account for the shape of the ground motion spectrum, thus resolving issues of bias when convolving vulnerability with hazard to estimate the seismic risk. While other improved IMs have recently appeared, the distinct advantage of S_{agm} is that the corresponding seismic hazard curves can be produced by using existing S_a ground motion prediction equations (Cordova et al., 2000). Thus, our unreserved recommendation for application will be the adoption of S_{agm} (Equation 1) for all high-rise classes using five or more logarithmically-spaced periods that cover the entire applicable range, spanning at least from the lowest to the highest first-mode period of the index buildings. This will increase fidelity not just for a single structure, but for the entire set of index buildings, without adding to the computational overhead.

5.2 *Record selection & scaling*

For the actual structural analyses of the index buildings, a number of ground motion records needs to be employed for each IM level. At present, there are two distinct approaches that can be followed, namely selection and scaling.

Record selection essentially involves forming an appropriate suite of ground motion records that represents the important characteristics of the seismic threat for each index building and at each intensity level. Some limited scaling is usually required to achieve a certain IM level. Still, for modern structures, catalogue restrictions may render the process impossible by requiring excessive scaling factors anyway. Overall, ground motion selection necessitates a knowledgeable user with adequate understanding of the seismological conditions at the region of interest. Online tools have appeared that help to automate selection, e.g., the PEER Ground Motion Database for shallow crustal earthquakes in active tectonic regions, (http://peer.berkeley.edu/peer_ground_motion_database/). However, even the PEER tool requires some degree of knowledge of seismic hazard in the geographic region of interest, such as the range of magnitude-distance pairs that will tend dominate hazard at various levels of IM.

A simpler approach appropriate to analysts who lack such knowledge is to select a single suite of relatively strong motions for all structures. These are subsequently scaled to the required target IM levels. This is the typical approach associated with Incremental Dynamic Analysis (IDA, Vamvatsikos & Cornell 2002). Still, scaling by S_a tends to introduce bias for excessive scale factors, as the change in spectral shape is largely ignored (Luco & Bazzurro, 2007). Thus, it is highly recommended that such an approach be paired with the use of S_{agm} as the IM, since it will significantly reduce the bias associated with scaling relatively weak ground motions to the collapse levels of modern high-rise structures.

5.3 *Nonlinear dynamic analysis*

Assessing the performance of the index buildings entails subjecting each one to the set of selected and/or scaled ground motion records at multiple levels of the intensity measure IM. For reasons of post-processing simplicity, it is suggested to perform all nonlinear dynamic analyses in stripes of given values of IM (Jalayer & Cornell, 2007) for each ground motion. When scaling the ground motions, this essentially conforms to the paradigm of an IDA (Vamvatsikos & Cornell 2002) that is applied using the same specific increments (or steps) of the IM level for all accelerograms. From each analysis, only a few EDP values need to be recorded for each individual story: Peak transient interstory drift ratios and peak floor accelerations.

6 LOSS ESTIMATION

Estimating the seismic loss for each index building is split into three distinct types, according to ATC-58 standards: (a) Structural losses, pertaining to damage in the load-carrying members of the structure, (b) non-structural loss for damage to non-load-carrying components, such as partitions, air ducts, piping systems, etc, and (c) contents' loss. All three types are assessed using component fragility functions parameterized on the two EDP types recorded during the analysis. Following the ATC-58 paradigm, at each IM-level, each component has a certain probability of being in any of its damage states, which is in turn associated with a probabilistic cost function. Summing up such costs over the entire structure yields the total loss (Porter et al. 2013a,b).

The main deviation from ATC-58 is that the methodology does not try to account for the entire inventory of building components in detail. Instead, a simplified catalogue is used including only the several components that most contribute to the loss. Specifically, we suggest using the 2 or so structural components that are most likely to contribute to loss, and the top 5 or 6 nonstructural components selected according to their contribution to construction cost. When estimating content vulnerability, we suggest considering only the 5 or so content categories that contribute most to content replacement cost in the building. Simplified assumptions are then employed to also include the perceived losses on elements that were neglected. In all cases, the probability of collapse is explicitly incorporated as suggested by Aslani & Miranda (2005), as collapse is assumed to cause instant loss of the entire building and its contents and can dominate vulnerability at higher IM.

7 UNCERTAINTY PROPAGATION

Loss estimation is fraught with uncertainty. It can be attributed to aleatory (naturally occurring) sources, e.g., due to ground motion, building population, material property or repair cost variability, but also to epistemic contributions, mainly due to our own approximations. The modeling and analysis choices are primary sources of epistemic uncertainty that need to be taken care of.

Our approach is two-fold. First, epistemic uncertainties that stem from modeling and analysis choices are considered to influence the distribution of EDP response and are then incorporated into the component- or story-level fragility results. Thus, they are accounted for, but they are not explicitly propagated to the final result. Uncertainties regarding the ground motion and building sample variability can be treated according to three levels of detail, directly paralleling the sampling resolution of the building population

If only a single index building has been used, it is advised to use the resulting vulnerability curve for the mean and adopt the overall dispersion implied by the HAZUS-MH results for a similar class of buildings (NIBS 2003). Such dispersion values, including both inter- and intra- building variability, can be derived from the raw HAZUS data using the methods described by Porter (2010). This is the simplest possible approach and it is obviously dependent on the diversity of buildings within the asset class of interest, compared with the diversity of buildings within the various building types treated by HAZUS-MH. If the asset class being examined is fairly narrow, such as all soft-story woodframe buildings with 3 or more stories and 5 or more dwellings, the dispersion implied by HAZUS-MH's models of large woodframe buildings may overestimate the dispersion in the asset class. Of course, there is no guarantee that HAZUS-MH does not systematically underestimate uncertainty.

The next best level of sophistication is applicable to three index buildings. The results from this limited sample can be used to derive both an overall mean vulnerability and the associated inter-building dispersion. By assuming the intra-building variability to be roughly equal to the inter-building result means we can provide an overall dispersion estimate by inflating the three-sample dispersion by a factor of 1.4.

When five or more index buildings have been analyzed, probabilistic methods can be employed to properly propagate the uncertainty. First-Order-Second-Moment concepts (Benjamin & Cornell, 1970) can be applied as the simplest approach. Still, full Monte Carlo simulation (MCS) using random or latin hypercube sampling provides the analyst with more confidence in the results. Despite being the most straightforward of all methods suggested so far, MCS is not advised unless accompanied with a certain minimum of experience.

8 CONCLUSIONS

The outline of the Global Earthquake Model analytical vulnerability guidelines for classes of high-rise buildings has been presented. Sampling of representative index buildings is achieved using either a subclass-partitioning mechanism or a star-design, coupled with the innovative moment-matching procedure. Subsequent evaluation of each building is built upon a hierarchy of methods, both for modeling, analysis and uncertainty propagation, following the general lines of the ATC-58 paradigm. Overall, the proposed guidelines try to strike a balance between accuracy and practicality, offering several different paths to accommodate different levels of expertise and resources available.

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